MAR 1 7 1934

Engineering News-Record

March 15, 1934

First Essentials to Recovery

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• • • Experience with the PWA system since last June shows convincingly that delays and tedious deliberation are fatal to success. The banker attitude that originated in the RFC has proved incapable of producing the necessary quick action. Details of procedure throughout need revision.

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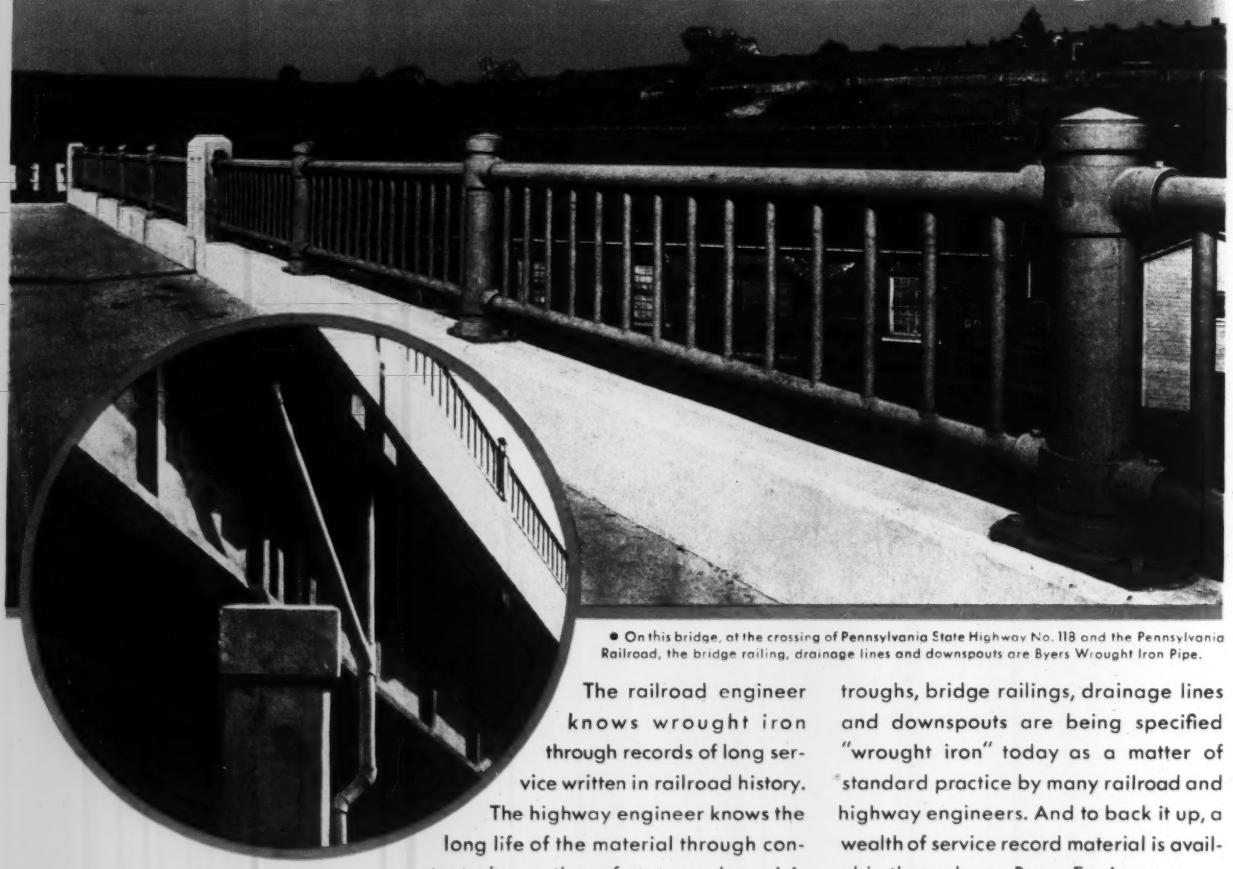

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In the News:

TRANSPORTATION COORDINATOR Joseph B. Eastman in his second report to the President urges that federal control, now exercised over the railroads through the Interstate Commerce Commission, be extended to highway and water transport. Drafts of proposed legislation to make such control effective accompanied the report.

SEVEN of the twenty PWA loan allotments made to limited-dividend corporations for low-cost housing have been rescinded, the local sponsoring bodies not being able to raise the equity required. The $23,670,500 involved has been transferred to the credit of the Public Works Emergency Housing Corp.

ANOTHER STEP in national planning has been made by the PWA in dividing the country into twelve regions, the grouping of the states being made from the standpoints of primary drainage, land use and transportation interests. District chairmen are being appointed for each region. Principal duties will be to assist the National Planning Board in coordinating state planning activities.

MELTING SNOWS AND ICE JAMS continue to foster floods. The Missouri River has caused extensive damage near Achison, Kan. An ice barrier above Safe Harbor Dam on the Susquehanna River caused flooding upstream as far as Harrisburg, Pa. Western New York State and Vermont were temporarily relieved of flood dangers by colder weather, but at East Brady, Pa., two 190-ft. spans of a four-span, through-truss bridge over the Allegheny River were carried out by ice.

ADDITIONAL FUNDS for construction of Golden Gate Bridge will be requested from PWA, it is understood. Preliminary discussion is expected to be followed shortly by the filing of a formal application. The original loan for financing the project was obtained from private banking interests.

In This Issue:

TWO WEEKS AGO on March 2, the first roller-gate dam and locks on the extensive slackwater canalization project for the upper Mississippi River were officially dedicated and opened at Rock Island, Ill. Twenty-seven similar structures are planned between St. Louis and the Twin Cities, and fifteen of them are under way. An article in this issue describes the construction of the locks, which included interesting cofferdam design and vibrated concrete. The dam construction will be described in the March 29 issue.

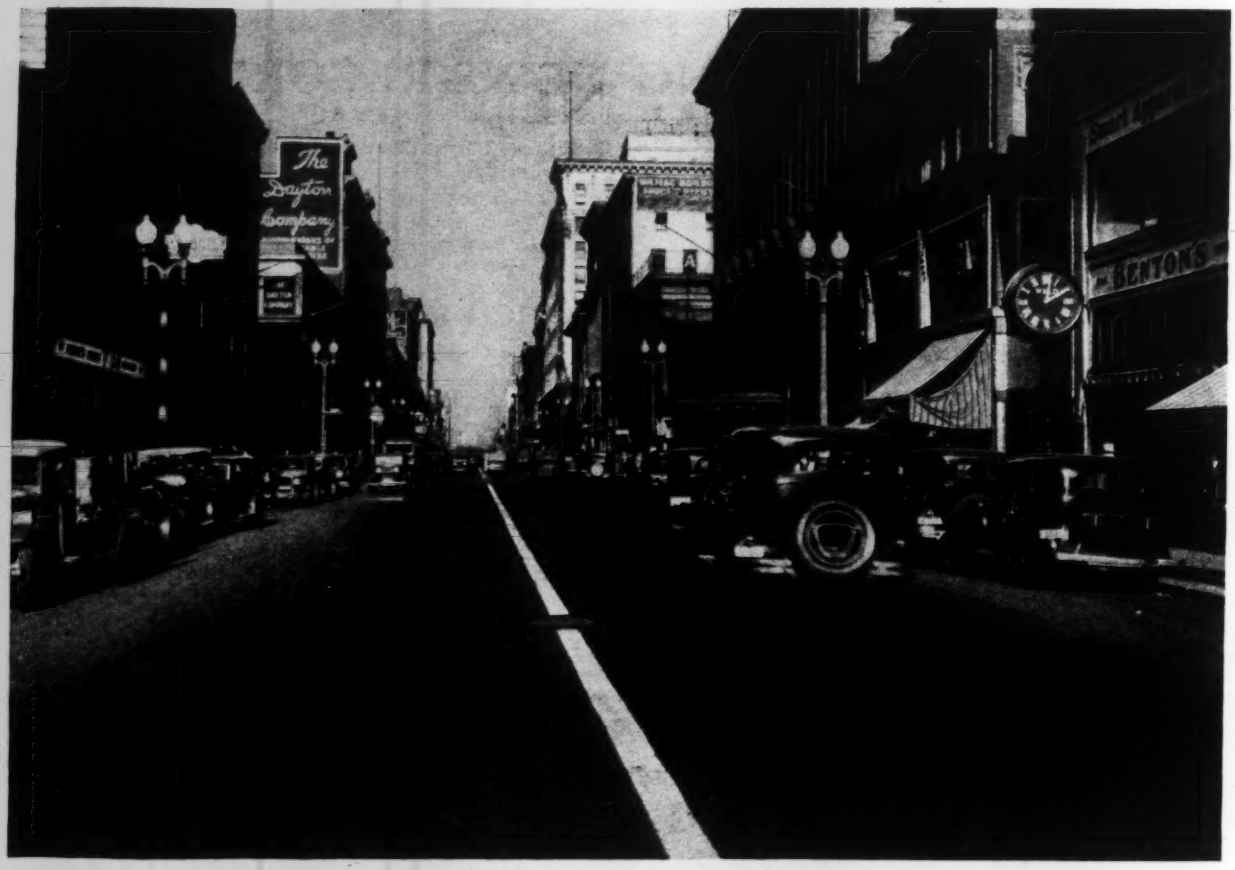
INDIANAPOLIS, in common with many other cities, has carried on street-paving work through the winter. High-early-strength cement and heated aggregates and water have been standard elements of the work. Also on one job a 6-in. layer of burlap and straw was wetted down and in freezing formed an airtight shield about the slab, as an aid in curing. Procedure on two typical jobs is presented in this issue.

IN THE relatively few years in which aeronautical engineers have been studying airplane design they have developed a vast amount of new information concerning wind forces. Civil engineers who could use this information have never had it available in a form that was usable or a language that was familiar. W. W. Pagon, consulting civil engineer, Baltimore, has been identified with many of these aerodynamic studies and has used the results in structural design. In this issue he presents the first civil engineering introduction to the modern work of aerodynamic engineers.

STREET RAILWAY COMPANIES are vitally interested in developing a track structure that is both smooth and low in maintenance expense. City officials are no less concerned with the results that they may achieve. Two designs of concrete-embedded track have recently been laid in Brooklyn, N. Y., in the hope of securing a more satisfactory structure than has previously been available.

FOR MORE THAN TWO DECADES reference to the St. Lawrence River Bridge at Quebec has meant only one thing—the famous railway cantilever whose remarkable design and collapses during construction comprise an indelible chapter in engineering history. It was also the only bridge over the river below Montreal. After next year this latter distinction will not exist, for a new bridge with a 1,059-ft. suspension span will cross the north channel at the Ile d'Orleans. The design of this structure, which includes interesting offshore anchorages, is given in an article in this issue.

THE PHRASE "prevailing rates of wages," has become a part of the language of construction since the beginning of NRA days. It was inevitable that it would be extended to encompass engineers' salaries, when an engineers' code should be formulated. As an aid to code authorities and others, an Am. Soc. C. E. committee has analyzed the salaries of more than 16,000 highway engineers and drawn up a wage schedule, weighted according to region, population and occupational conditions.



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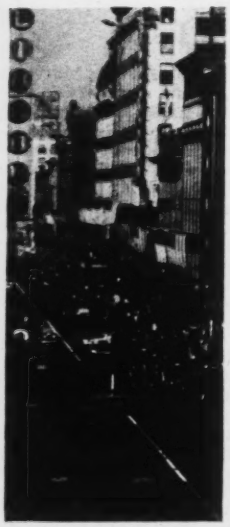

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ENGINEERING NEWS-RECORD

New York

March 15, 1934

FIG. 1—AUXILIARY LOCK at Rock Island nearing completion with gates in place. View upstream shows land wall of main lock at right.

Lock Concreting at Rock Island Dam

First of the new canalization works for the upper Mississippi River introduced new designs of cofferdams and inaugurated compaction of concrete by vibration, since specified for all lock work

AN OUTSTANDING introduction to the structure forming the 9-ft. navigation improvement of the upper Mississippi River (*ENR*, March 8, 1934, p. 322) is furnished by the locks and dam placed in service a week ago at Rock Island, Ill. (officially, "Dam 15"). They are the first finished works in the new project; the dam is the largest roller-gate dam in the world. With the sewer-containing seawall, to hold the pool back from the Davenport, Ia., waterfront, the gate piers and sills and the construction of the auxiliary as well as the main lock, the concrete work is of exceptional variety. In construction characteristcs the work thus divides into two quite distinctive types of operation: (1) heavy concrete construction in locks, piers and seawall; and (2) roller-gate structure and construction. The locks and seawall are described in the following article. A succeeding article will describe the dam construction and roller-gate erection.

Lock structure

The local geography of Rock Island, Moline, Arsenal Island and Davenport is shown by Fig. 2. As shown, the river loops west in its general course south and places the Illinois and Iowa cities and Arsenal Island on an almost direct north and south line. The dam crosses the river at an angle of about 30 deg. with this line, and the locks are on Arsenal Island at the south end of the dam. As bearing on lock construction, the location of considerable parts of the structure under the road and railway bridge to Davenport is to be noted. The seawall along the Davenport waterfront and the levees on Arsenal Island and fronting Bettendorf are structures required by the new water level created by the pool formed by the dam. The old navigation and power structures are to be noted in passing.

Walls and Gates—The standard lock plan for the upper Mississippi 9-ft. channel calls for a main and an auxiliary lock at each dam. At some of the locations, as Dams 4 and 5 (*ENR*, Jan. 25, 1934, p. 101), only the main lock and the upper-gate masonry of the auxiliary lock are being built at first, but at Rock Island both locks have been constructed as shown by Fig. 3. In general, the lock structures and equipment are typical of those for all the twenty-odd locks planned.

As indicated, the walls of the locks are founded on rock, a limestone, which at this point forms a local anticline. The lower guidewall on the land side is founded on a rockfilled crib placed in a trench that was excavated in the wet.

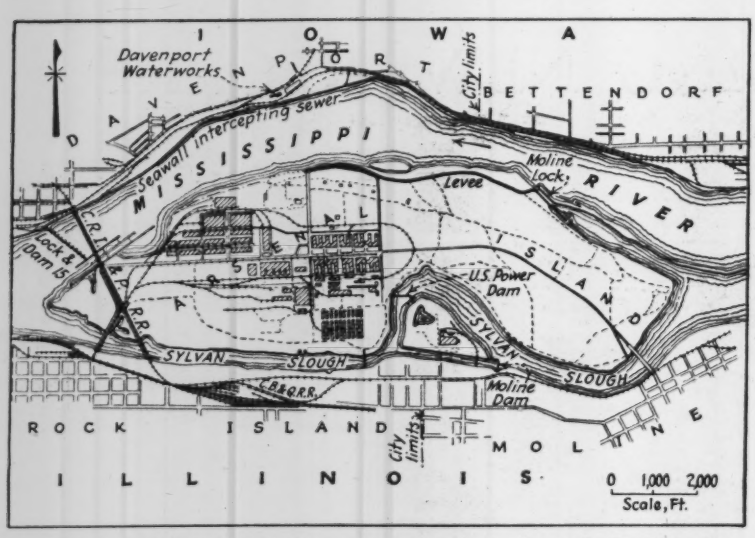


FIG. 2—LOCATION MAP of structures of canalization works at Rock Island which include the largest roller-gate dam ever constructed.

Similarly, the river guidewall downstream is placed upon a concrete-filled crib into which the concrete was tremied. These cribs are 20 ft. wide by 14 to 16 ft. high, and the tops are slightly more than a foot above extreme low water.

The completed locks are gated by four leaves of structural steel weighing approximately 80 tons each, two leaves to a set, forming mitre gates meeting at the center and angled upstream. Each lock is provided with poiree dams above the upper and below the lower mitre gates which, in conjunction with the tainter valves, cofferdam off any lock chamber and permit gate repairs at any time.

Lock Equipment—The mitre gates are operated by special electrical powered and controlled machines from power generated in a 312-kva. generator mounted on the river lock wall. The water level in the locks is controlled by tainter valves: four to the main and four to the auxiliary lock. The tainter gates are mounted in chambers built into the gate blocks to which the mitre gates are hinged. The tainter valves control the water passing through the intake and outlet ports and tunnels in the walls leading from the upper pool to the lock chambers, and from the lock chambers to the lower pool. These gates are operated by chain hoists actuated electrically from the power house.

The machinery-operating chambers in the walls are covered with electrically welded grating, and the tops of the mitre gates are provided with subway grating walks. Galvanized iron railings surmount all walls. Each lock has mosaic tile water-level gages, showing the stage of the river in the upper and lower pools. On the intermediate wall is mounted a combination warehouse and panel-board room, required because of the scarcity of space in the power house, which is mounted in the river wall.

The locks will have ample illumination standards up and down their length. The tops of the walls are provided with cast-steel snubbing posts, and tow-hitch wall inserts at convenient intervals provide, vertically, for tying up tow boats at any elevation of the water. The up- and downstream noses of all cut waters have heavy steel-plate armor to prevent ice erosion.

Lock construction

At the lower end of Arsenal Island is the Arsenal Bridge, Fig. 2, connecting Illinois and Iowa. This bridge spans the locks and pool formed by the new dam by five 250-ft. spans and one swing span 320 ft. long. Through this swing span all river traffic passes, and the locks pass through the swing-span area. To maintain the traffic through the swing-span area, through which the locks also passed, it was necessary to divide the work of lock construction into two sections; first, the river wall with its upper and lower guidewalls and the intermediate wall; and second, the river wall with its lower guidewall.

The space occupied by the lock structure is about 17 acres. The over-all length of the land wall is 2,468 ft., of the river wall 1,371 ft., and of the intermediate wall 1,331 ft. The height of wall above low water is 26 ft. There were about 140,000 cu.yd. of concrete, 2,000,000 lb. of structural steel, 65,000 cu.yd. of rock excavation, 54,000 cu.yd. of common excavation, 1,200,000 lb. of steel castings embedded in the face of the walls to protect the concrete against ships, and 52,000 ft. of electrical conduit buried in the walls. About 4,200 lin.ft. of 20x20-ft. cofferdam was built, involving the use of 2,000,000 lb. of sheetpiling.

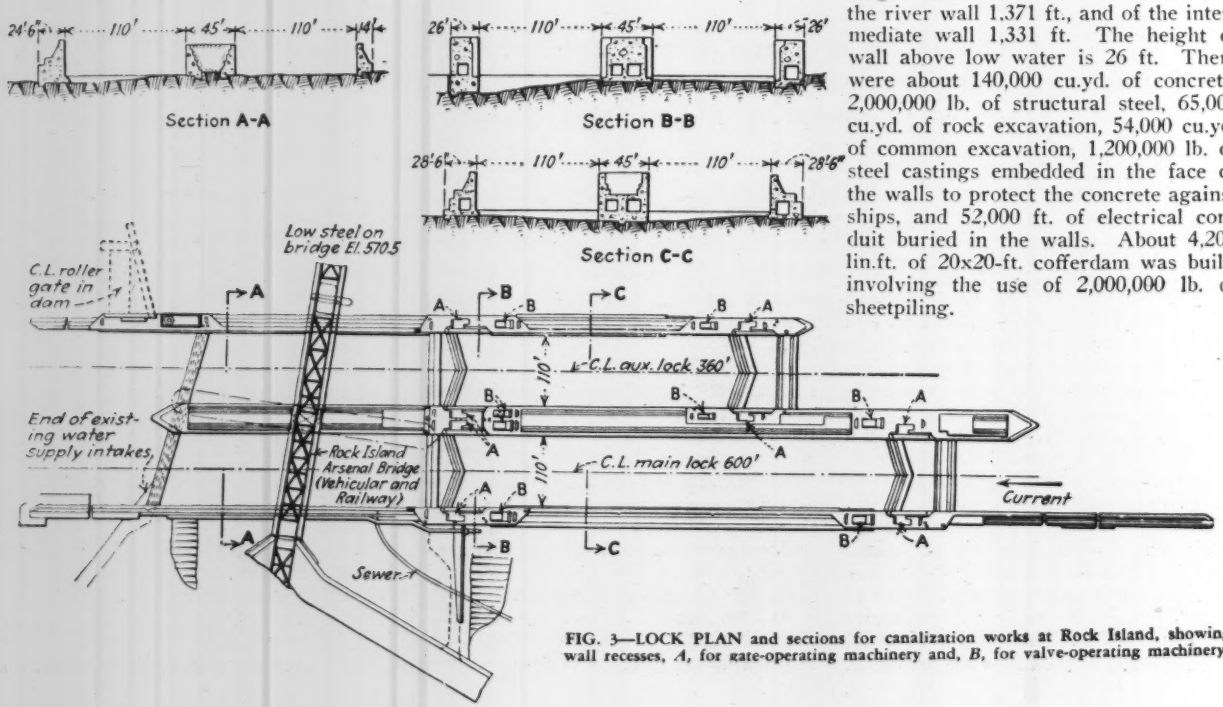


FIG. 3—LOCK PLAN and sections for canalization works at Rock Island, showing wall recesses, *A*, for gate-operating machinery and, *B*, for valve-operating machinery.

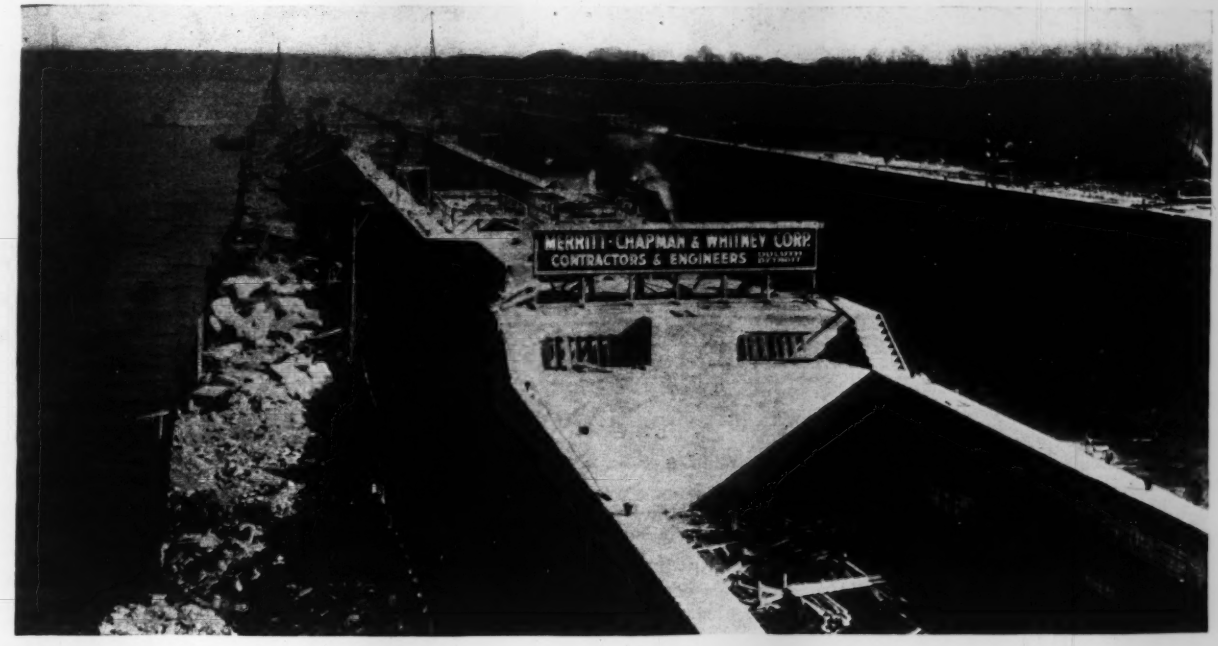


FIG. 4—SHEETPILE COFFERDAM and intermediate lock wall of twin walls to be filled between with earth. Note solid "gate block" portion of wall.

Unusual Cofferdam—The land and intermediate walls were constructed in one cofferdam which inclosed the swing-span pier, and river traffic was diverted into the channel that is now the auxiliary lock. The first cofferdam was built to present a true line to guide shipping through the north half of the swing-span. When the land and intermediate walls, forming the main lock, were finished, traffic was routed through that channel and the second cofferdam was built in which to construct the river wall. Thus the traffic condition made two cofferdam constructions necessary.

Two types of cofferdam were developed in the construction of the locks and dam. One type consisted of two rows of steel sheetpiling (Fig. 4) driven to refusal in the muck overlying the bed rock and guided into place by a floating frame. The tops of the piles were waled with 10x16-in. timbers and tied across by 1¼-in. bolts, 10 ft. on centers. At the elevation of the water at the time these piles are driven is floated into place another pair of wales of the same size, and these two wales are connected across the cofferdam by the same size bolts on the same spacing.

Every seventh pile in each row has connected 3, 4 or 5 ft. from its bottom end, a 1-in. cable rendered through holes burned in the pile and clipped together over a circular casting on the outside. The upper end of this cable is welded near the top of the pile, and the piles similarly equipped are driven opposite each other. After the floating guide-frame or form has been moved ahead, the cables are cut loose from the top end, rendered diagonally across the cofferdam and made fast to an eye-bolt with long threads, which is entered from the inside of the cofferdam, angling upward through a resting casting and

FIG. 5—COFFERDAM INCLOSURE at downstream end, showing floating mixing plant, section of wall forms and track system for concreting trains and cranes.

FIG. 6—LOCK PIT, showing swing span through which locks pass, land mixing plant and section of wall forms.

tightened with ordinary bridge wrenches at the time the fill is placed. This construction enables cofferdams to be built in fairly deep water without internal bracing, at a great saving in cost.

The other type of cofferdam was an adaptation of the Ohio type. The Ohio cofferdam consists of wood frames built upon barges and hinged at their ends in such a manner as to allow the lowering of a continuous frame to the river bottom. On the inside of this frame are placed ordinary 2x12-in. planks lightly tacked, and the fill is placed immediately to hold the sheeting out against the frame. This is a fragile construction and requires the banking of both sides with heavy fill to prevent the washing away of the finer particles when being built. It requires a very large fill, but while it is cheap to build as a structure, it has no salvage value, for when a cofferdam is wrecked the timbers, rods and bolts are chewed up in the operation of removal.

The idea was conceived of making a structural-steel framework in such a manner that the ends would interlock and still be free for removal, lagging the inside of the frame with two layers of loose boards and filling between in the normal manner, doing away entirely with the outside bank fills common to the Ohio type. Instead of using heavy filling material, sand was used without a distortion effect upon the steel, and as a consequence the cofferdam has a nearly 100 per cent salvage value.

Excavation and Concreting—The excavation in the main lock was by steam shovel, dragline, cars and dinkeys, hauling the spoil to the esplanade area back of the land wall, where is was shaped into a fill, which was topped off with a foot of black loam to permit the growing of lawns. The excavation in the auxiliary lock was handled by draglines disposing of the spoil directly into the space provided in the intermediate wall under which two tunnels run, one for the main and one for the auxiliary lock.

All bedrock was grouted under pressure after completion of the concreting and drained by drain holes and suitable box drains to control hydraulic uplift. The walls were constructed in blocks or monoliths about 30 ft. long. The monoliths were separated by common roofing paper and asphalt-fiber strips for contraction and expansion. No waterproof copper seals were inserted between the monoliths. The specifications limited the pouring to not less than 4 ft. nor more than 18 ft. at one operation. This divided the wall vertically into four lifts. The lower lifts varied from 4 to 24 ft., conforming to the excavated bedrock structure. The second lift inclosed the tunnel walls. The third lift capped the tunnel, and the fourth lift built the top. Concreting operations were maintained at the rate of 700 cu.yd. per pouring day, or an average of 19,000 cu.yd. per month.

Both land and floating concrete plants were served by derrick scows which unloaded the material from barges into steel bins over the mixers. The floating plant had a 1-yd. tower from which a boom chute was suspended, allowing those parts of the work adjacent to the cofferdam to be reached directly. The land plant furnished concrete into standard 1-yd. bottom-dump buckets, which were hauled to position by cars and dinkeys and the concrete deposited into the forms by cranes.

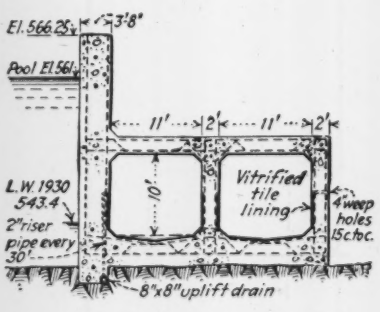


FIG. 7—SECTION of seawall and sewer along waterfront of Davenport, Ia., across the river from Rock Island.

The contractor for the locks was the Merritt, Chapman & Whitney Corp., Duluth, Minn., a subsidiary of the Merritt, Chapman & Scott Corp., New York, N. Y.

Seawall and sewer

In contracting the various parts of the work at Rock Island, the seawall and sewer (Fig. 2) were made a part of the general contract for the dam, which was awarded to S. A. Healy, Detroit, Mich. With the piers and sills for the dam, which are more appropriately discussed in the article on dam construction, the seawall and sewer constituted the main concreting operation of the contractor for the dam.

Briefly, as a part of the plan for the Rock Island pool, an intercepting sewer was planned to traverse the waterfronts of Davenport and Bettendorf and to carry the sewage, formerly discharging directly into the river, to an outlet below the new dam. On the part of the riverfront where a seawall was required to hold the pool level back from the waterfront property, the sewer was incorporated in the seawall. The wall and sewer sections varied at different places, but the general type of structure at one of the larger sections is shown by Fig. 7.

The construction of the seawall and sewer is of principal interest because of the experience with vibrators for consolidating the concrete. As reported by Major R. A. Wheeler, district engineer, Rock Island, Ill., the use of vibrators was not required by the specifications, and they were used, therefore, only when the contractor considered them advantageous. In general, the resulting experience and opinion were about as follows:

The sewer was about 4 miles long, the lower 2 miles of which were heavily reinforced structure comprising a retaining wall and double sewer openings, each 10x11 ft. in size. The front, or

retaining wall, was 4 ft. thick, while the middle and rear walls and the roof slab were 2 ft. thick. The structure was poured as a monolith above the inverts, except the retaining wall above the tops of the sewers.

It was not possible to work men in the narrow forms with the heavy reinforcement, and since the depth of the forms was too great for satisfactory spading from the top, the use of internal vibrators was indicated. The vibrators used consisted of an eccentric rotor inside a steel barrel about 3 in. in diameter and about 30 in. long. This was connected by a flexible hose containing a driving chain to a small electric motor which was handled from point to point on top of the forms and connected to the lighting circuit. The vibrations were 3,600 per minute. The concrete contained six bags of cement per cubic yard with a slump of approximately 3 in. Crushed stone was used as coarse aggregate.

Steel forms were used for the openings of the sewer, and the vibrators did not prevent the formation of air or water pockets on the steel surfaces, resulting in a pitted or "pockmarked" finish. This blemish was partly corrected by the use of light, long-handled spades along the faces of the forms.

Some experience and observation in the use of the vibrators are necessary to secure the best results. It was found inadvisable to leave the vibrator too long in one place, as a pocket of sloppy mortar would form around it giving the appearance of segregation. This probably meant no more than that more water and cement paste were present in the mix than were necessary if the entire mass were as thoroughly vibrated. However, on this work no attempt was being made to reduce the amount of water and cement used with the object of securing a concrete of given strength at less cost. Rather, the end sought was properly to compact a given mix under conditions that did not admit of hand-placing. It was found that 30 to 45 sec. was sufficient time to leave the vibrator in one spot, after which it was moved a few feet distant. This method of operation appeared thoroughly to compact the mass.

Experience gained on this work with the internal vibrators indicated that a concrete mix of stiffer consistency than is practicable to work by hand methods can be satisfactorily compacted with their use.

Engineering direction

The construction described was carried out under the Rock Island District, Upper Mississippi Valley Division, U. S. Army. At the start of the work Geo. R. Spalding, Colonel, Corps of Engineers, was division engineer. He was succeeded by E. L. Daley, Lieutenant-Colonel, Corps of Engineers. R. A. Wheeler, Major, Corps of Engineers, is district engineer at Rock Island.

Review of Soil-Bearing Test for Columbus Water Tanks

Failure to follow basic principles of soil mechanics vitiates test results on several important counts

By F. S. Besson

Lieut.-Colonel, Corps of Engineers, Washington, D. C.

IF SMALL-SCALE soil-bearing tests are to be applied with any reasonableness to actual load conditions, the basic principles of soil mechanics have to be followed. As an example of the failure to do this, the soil-bearing tests for the Columbus, Ohio, water tanks (*ENR*, Oct. 26, 1933, p. 500) deserve attention. The method of conducting the tests is indicated by Fig. 1.

There are two distinctly different types of settlements: first, that due to consolidation of the foundation soil; second, that due to lateral flow of soil from under the structure. Total settlement may be due to a combination of the two types.

Consolidation—If the voids of the soil contain only air, consolidation can take place immediately with the application of the load, because air may readily be squeezed out of the voids. Most soils, however, contain water, and a decrease in volume can be forced only by squeezing out water. Accordingly, the rate of consolidation depends upon the permeability of the soil through which the excess water has to escape. With pervious sands, complete consolidation may take place almost immediately, whereas with clays, on account of their imperviousness, total consolidation under light loads may not take place for a considerable period of time, and under heavy load not for a long period of years.

In the test, Fig. 1, the base of the oak pedestal is 0.4 sq. ft. Under unit load equal to that which the actual structure was designed to carry, settlement amounted to about 0.02 ft. To account for this settlement, only 0.008 cu.ft. of water had to be squeezed out of the voids in the soil. Under the abnormal conditions shown in the figure, with a sump 4 ft. deep adjacent to the loaded area (conditions contrary to those under which the actual load was to be carried) 0.008 cu.ft. of water could easily be squeezed from the soil, and on this count alone the test is of doubtful value.

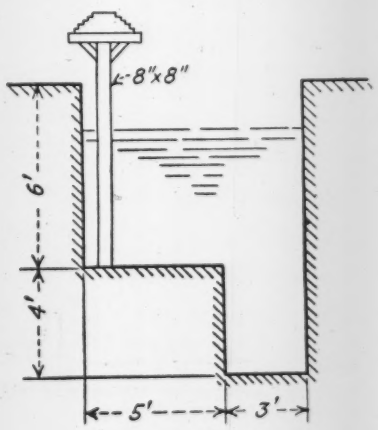


FIG. 1—METHOD of conducting soil-bearing tests for Columbus water tanks

Lateral Flow—Many soils, when wet, are in a plastic condition, and, when loaded, the soil passes from under the loaded area in viscous flow. This displacement accounts for the bulges that sometimes occur on the ground surface, surrounding structures built on plastic foundation soil.

The vertical wall of the sump cannot be stable, and with the shelf under load the vertical wall, 4 ft. high and 3 ft. wide, could readily bulge to include a volume of 0.008 cu.ft. without detection, though subjected to careful inspection. This factor adds to the doubtfulness of the value of the test.

Size of Loaded Area—When pure clay is the foundation soil, the coefficient of cohesion is very large. With unit loads well below the ultimate bearing capacity, the area-settlement diagram, due to consolidation, approaches a straight line with the equation

$$S = CW'q$$

where S equals the settlement, C a constant for the particular soil under test, W' the width of the area loaded and q the unit loading. To compare settlements for two loaded areas, the areas must be geometrically similar.

In other words, for a given settlement with a pure clay foundation the unit load on a large area is but a fraction of that on a small area, as long as the loads are well within the ultimate bearing capacity of the soil. However, this differential decreases as the ultimate bearing capacity is reached, at which stage, as indicated in Fig. 2, the unit load for a large area is not much smaller than for a small area.

When pure sand is the foundation, the soil is practically cohesionless. With small unit loads, the settlement for equal unit loads is independent of the size of area loaded. However, with larger unit loads the settlement is less for a large area than for a small area, and, as indicated in Fig. 2, the ultimate bearing

capacity of a large area is several times that of a small one.

Soils having coefficients of cohesion intermediate between pure clay and pure sand have intermediate properties.

Some of the foundation piers under the actual load, investigated in connection with the test shown by Fig. 1, were designed with bases 500 times larger than the base of the pedestal shown n the figure. A single test area, so small in comparison with actual conditions, is not sufficient to establish a relationship that will permit satisfactory estimating of actual load settlement based upon test settlement.

Depth Below Surface—In addition to considering the size of the area loaded,

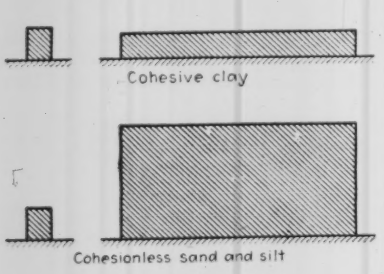


FIG. 2—TYPICAL bearing capacities of large and small areas for a given settlement.

values for admissible soil pressures must consider the depth of the loaded area below the surface of the ground. Settlement S_d, at depth d below the ground surface, is but a portion of S which occurs when the loaded area is on the surface.

With d the depth and W' the width of the area subjected to a given load, the ratio $\frac{S_d}{S}$ varies with $\frac{W'}{d}$.

To illustrate this relationship, assume that an area with $W' = 3$, under a given unit load, settles S at the surface of the ground. Assume, for the particular soil, that $\frac{S_d}{S} = \frac{1}{2}$, when $d = 5$. Under these assumptions the ratio $\frac{W'}{d} = \frac{3}{5}$.

If W' were changed to 16 ft. and d to 10 ft., the ratio $\frac{W'}{d} = \frac{8}{5}$; and then the ratio of the settlement S_d at depth 10 ft., for this area, to the settlement S at the surface for the same area would be greater than ½, or $\frac{S_d}{S} > \frac{1}{2}$. The ratio $\frac{S_d}{S}$ approaches unity as $\frac{W'}{d}$ is made larger and larger.

If W' were changed to 2 ft. and d to 20 ft., the ratio $\frac{W'}{d} = \frac{1}{10}$, and then

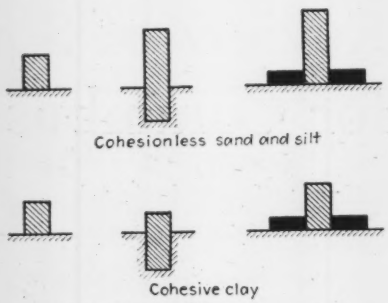


FIG. 3—TYPICAL EFFECT upon bearing of depth below surface and surcharge.

$\frac{S_d}{S} < \frac{1}{2}$. Settlement S_d becomes a smaller and smaller portion of settlement S as $\frac{W'}{d}$ decreases in value.

The variation of $\frac{S_d}{S}$ with $\frac{W}{d}$ is greater for sands that lack cohesiveness than for clayey soils of high cohesiveness.

To make comparison of an area A'' at a depth d'', with a small geometrically similar test area A', the test area should be at depth d' such that $\frac{S_d}{S}$ is maintained constant—that is, $\frac{W''}{d''}$ should equal $\frac{W'}{d'}$.

The effect of depth may be shown by the two left-hand diagrams in Fig. 3, which illustrate that, for a given settlement, the bearing capacity of an area below the surface may be greater than for an area on the surface of the ground. These diagrams also illustrate the greater effect had with sands than with clayey soils.

In the test represented by Fig. 1, $\frac{S_d}{S}$ is far from that for actual designed conditions. With the arrangement shown in the figure, with a sump on one side of the pedestal and on the other side a vertical bank, it is even impossible to determine a correct value for the test ratio $\frac{W'}{d'}$.

Failure to Follow Principles—Thus the test, in that it does not follow the basic principles of soil mechanics, fails to give satisfactory results on four counts:

1. It does not permit proper weight to be given the factor of consolidation.
2. It does not permit proper weight to be given the factor of lateral flow.
3. It does not establish the relationship between size of test area and size of actual load area.
4. It does not maintain the ratio $\frac{S_d}{S}$ constant.

Two Winter Paving Jobs in Indianapolis, Ind.

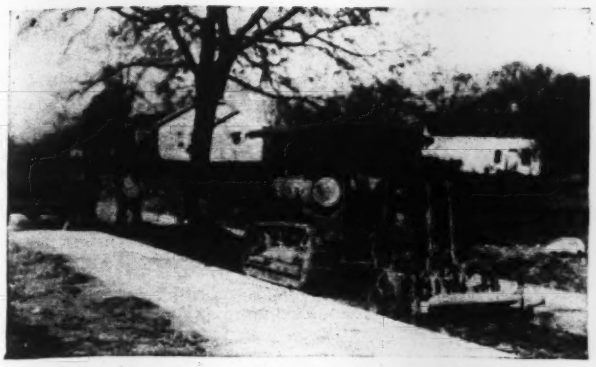
CITY street-paving operations carried on this winter in Indianapolis, Ind., are notably characteristic of municipal operations. One operation was strictly a resurfacing job, and the other was new paving of a wide street leading out of the city and continuing as part of a main trunk highway. Both operations, too, included coincident subsurface improvements, in one case a sewer and in the other the installation of services for water and gas. In this account of the work only the concrete paving is described.

Resurfacing Ohio St.

Included in the late fall and early winter construction program of Indianapolis was a four-block concrete resurfacing project that was let and completed in December. The street resurfaced was Ohio St., originally paved with wood block on a concrete base. The wood block was stripped from the base, and a new surface of reinforced concrete was laid to a minimum depth of 5 in., but averaging 6 in. Before laying the surface a sand cushion was placed on the base. The first resurfacing concrete was placed on Dec. 16, and the last on Dec. 24.

The length of the resurfacing was 1,200 ft., and the width 27 ft. It was reinforced with 67-lb. mesh. No mix was specified, but the concrete was required to develop 3,500 lb. compressive strength in 28 days. The city's requirement for standard concrete paving, 7 in. thick, is 4,000 lb. in 28 days. The contractor's bid price for the work was $5.10 a linear-foot, using regular cement and $5.51 a linear-foot, using high early-strength cement. The square-yard equivalents of these prices are $1.70 and $1.87 respectively. Bearing in mind, however, the lateness of the season, and with a view to reducing to a minimum the hazards of cold weather, the city decided in favor of the latter bid.

Air temperatures were normal for this season of the year except for a few days when the thermometer failed to rise above 40 deg., and the minimum night temperature was 17 deg. Tests revealed an average three-day strength of 3,800 lb. and an average seven-day



SCARIFYING is the first step in preparing an 8-ft. subgrade for side strips of 56-ft. Indianapolis pavement.

ROLLING and templeting of 8-ft.-wide strip subgrade after scarifying and excavation are completed.

strength of 4,800 lb. (The opening requirement for the pavement was 2,000 lb.) Also, the minimum day temperature recorded for the surface of any of the slabs under their burlap and straw covering was 54 deg.

Premixed concrete was used on the work, being delivered in transit-agitated trucks from a commercial plant about 4½ miles from the job. The mixing water and the aggregates for the concrete were heated, and despite the long haul they registered a temperature of 64 to 70 deg. when deposited.

Fifteen men were directly employed on the paving work. In addition, there were five men engaged at the plant supplying the ready-mixed concrete for the job, and eight truck-drivers delivering the concrete. The contractor was R. M. Bowen, Indianapolis, Ind.

Paving Lafayette Pike

The first NRM project to be let in Indiana was for the concreting of 2,350 lin.-ft. of the Lafayette Pike in Marion County, between 16th St., Indianapolis, and Cold Spring Rd. To carry on operations through the winter with a minimum of delays from cold weather, high-early-strength cement was specified for all but a fraction of the yardage.

The paving consists of 1,125 lin.-ft. of 9:7:9-in. concrete slab 40 ft. wide, and 1,225 lin.-ft. of 7:9:7-in. concrete slab 56 ft. wide. All of the wider slab and half of the other are within the Indianapolis municipal limits. The first concrete for paving was placed Dec. 3.

The slabs for both widths have a longitudinal center joint of submerged steel plates with dowels. Single ¾-in. bars provide reinforcement at the sides. Transverse expansion joints with 1-in. filler extending the full depth of the slab are installed at intervals of 105 ft., and transverse joints (contraction) with ⅝-in. filler submerged 2½ in. are installed at intervals of 35 ft.

The total concrete in the paving is 2,400 cu.yd., and the mix is about 1:2:3. The opening requirements for the concrete are a beam strength (modulus of rupture) of 550 lb. and a minimum age of three days. This strength was actually developed in from two to three days.

The 40-ft. slab is being laid in half-widths, and the 56-ft. slab in two 20-ft. and two 8-ft. widths, the latter at the sides. The 20-ft. widths are machine-finished, while the 8-ft. widths are finished by hand. Concrete is delivered in 4-yd. batches by transit-agitated trucks from a commercial mixing plant located about two miles from the job.

Cold weather precautions for the concrete, in addition to the use of high-early-strength cement, include heating the aggregates and mixing water and then covering the placed concrete with burlap and straw to a depth of 6 in. This covering is wet down in order to freeze and form an airtight shield about the concrete. The temperature of the concrete at the mixing plant has averaged 75 deg.

The thermometer registrations at the surface of the concrete 24 hours after each pour have shown much less difference from the mixing temperature than would be expected in view of the long haul and frequent low air temperatures. Thus at the end of one 24-hour period, during which air temperatures had ranged from 17 to 50 deg., thermometer readings taken at various points under covering showed a minimum of 60 deg.—a difference of only 15 deg. from that at the mixer.

The grade is also covered with straw as soon as it is finished and this covering is maintained pending concreting.

The contractor has been operating seven days a week on this project, employing a total of about 100 men working in two 4½-hour shifts a day. The paving shifts number 30 men, curb and gutter shifts 10 men, and grading shifts 10 men. In addition to these forces of the contractor, 15 men are engaged at the mixing plant and in transporting the concrete to the job. From six to seven trucks are being used.

The paving was let to contract by the state as an NRM project. H. D. Sommers & So., Indianapolis, is the contractor.

DOWEL BARS at contraction joints are carefully supported and aligned.

AGITATOR TRUCKS pour preheated central-plant-mix hauled 4½ miles.

What Aerodynamics Can Teach the Civil Engineer

Aeronautical research has revealed many facts which point to the development of a consistent theory of wind forces and their action on structures

By W. Watters Pagon
Consulting Engineer, Baltimore, Md.

CIVIL ENGINEERS concerned with the design of structures to resist wind have in general taken surprisingly little interest in the subject of wind forces themselves and have been content to abide by the results of tests made 75 to 100 years ago by such men as Duchemin. There has been much discussion, it is true, of the strict design of the steel frames of buildings to resist wind and earthquake forces; there has been extended argument as to whether 30 or 20 lb. per sq.ft. should be used and whether the projected area should be multiplied by 1½ or 2 to allow for the leeward truss of a bridge or a tower, and whether high buildings should be figured for 40 lb. per sq.ft. But in the meantime tropical hurricanes have brought periodic destruction to the southern states; tornadoes have wrecked large portions of cities in the Mississippi Valley; thunderstorms have unroofed houses in almost every state; wind and sleet storms have played havoc with telephone and transmission lines. And such storms are still largely unknown quantities because we have not had the foundation upon which to build a consistent theory of wind forces that will correlate all of the phenomena.

In the meantime, the aerodynamic engineer has been making a multitude of tests and doing an immense amount of theoretical study, of which the civil engineer is grossly ignorant. The writer, because of his many contacts with the aerodynamic field, has felt an obligation to make available to the civil engineering profession of which he is a member some of this research which has resulted in much important fundamental knowledge.

Why did the Meyer-Kiser Building twist during the Miami hurricane? Why do roofs blow off in thunder squalls, when common practice says there is pressure on them? What difference in design pressure is justified between Atlantic City and Baltimore or Philadelphia? How does pressure vary with height above the ground? Is there any basic theory by which a tornado may be studied? What is the internal structure of a thunderstorm? Many such questions can be asked, and aerodynamics can already give valuable aid in answering them.

Aerodynamics, because it is a modern science, has a heritage of mathematics and test that few of the older sciences possessed. It has been fostered on the theory of similitude, nurtured by able mathematicians, and now in adolescence is being brought up on a basis of rigorous and extended tests. At Langley Field, for example, a full-size airplane may be tested complete in a huge wind tunnel and compared with small-scale models.

Similitude and Reynolds number

Consider for a moment what similitude means. The dimensions of a building model have a given ratio to that of the prototype. The velocity through a turbine model has a given ratio to that of the turbine itself. In each case there is a scale ratio. This ratio, which is purely a number, need not be a ratio of feet to feet, or feet per second to feet per second. It may be a ratio of any number of quantities so long as their dimensions cancel out. For example, velocity pressure in aeronautic practice is

$$\frac{wv^2}{2g} = \tfrac{1}{2}\rho v^2,$$

where ρ is the density or pounds per cu. ft. (w) divided by the acceleration of gravity (g). Then

$$\tfrac{1}{2}\rho v^2 = \text{number} \times \left(\frac{\text{lb.}}{\text{ft.}^3}\right) \times \left(\frac{\text{ft.}}{\text{sec.}}\right)^2 \div \frac{\text{ft.}}{\text{sec.}^2} = \text{number} \times \left(\frac{\text{lb.}}{\text{ft.}^2}\right) = \text{unit pressure.}$$

Also viscosity, μ, is the unit frictional force which will cause two parallel planes 1 ft. apart to move past each other at the rate of 1 f.p.s. or it has the dimension

$$\frac{\text{lb.}}{\text{ft.}^2} \times \text{ft.} \div \frac{\text{ft.}}{\text{sec.}} = \frac{\text{lb. sec.}}{\text{ft.}^2}.$$

If we let D represent some (any) dimension of a body, e. g. the diameter or the length, then

$$\frac{vD\rho}{\mu} = \left(\frac{\text{ft.}}{\text{sec.}}\right) \times (\text{ft.}) \times \left(\frac{\text{lb. sec.}^2}{\text{ft.}^3\ \text{ft.}}\right) \div \frac{\text{lb. sec.}}{\text{ft.}^2} = \text{number.}$$

This number, or ratio, is fundamental in aerodynamics and is slowly being recognized in applied hydrodynamics. It is "Reynolds' number," commonly called R. Since, $\mu/\rho = \nu$ (called the "kinematic viscosity") $R = \dfrac{vD}{\nu}$. (1)

If feet and miles per hour, or feet and feet per second, or meters and meters per second, or any other systems of units, be used throughout, the value obtained for R is the same, because R has no dimension. It is a pure number.

To many engineers Reynolds' number is a pure abstraction, like entropy in thermodynamics. But truly it has a definite meaning. Since we can divide and multiply by the same quantity without affecting the value of R, then

$$R = \frac{vD\rho}{\mu} = \frac{\tfrac{1}{2}\rho v^2 D^2}{\tfrac{1}{2}\mu vD} = \text{number} \times \frac{\text{(unit pressure)} \times \text{(area)}}{\text{(viscosity)} \times \text{(velocity)} \times \text{(dimension)}} = \frac{\text{pressure forces}}{\text{friction forces}},$$

keeping it in mind that D represents any given scalar dimension. This indicates that the criterion which tests have shown to be so fundamental in describing the character of flow through pipes or about bodies is a simple ratio between the pressure force and the frictional force (but it is not a coefficient of friction). Once D has been assigned to some particular dimension, e.g. the diameter of a pipe, of a cylinder, of a sphere, the length of a flat plate, or the span of an airplane wing, regardless of how large or small it is, then R has a significant numerical value that will hold regardless of size. Hence two similar bodies at similar speeds, so that R is the same for both, will behave identically. This is all that R means.

It is not necessary to use R in the day's work of the civil engineer, but the idea of similitude has made applied aerodynamics possible and has permitted correlation of tests of all sizes, in all countries, because R has no dimension and no nationality.

Reynolds' number, however, applies in general only to rounded bodies, such as a chimney or a tank. It does not apply to bodies that have sharp edges, such as buildings and bridges, where the character of the flow is affected by these edges. This is no defect in Reynolds' number but is the result of another significant phenomenon that arises from friction,

namely eddy formation, and is closely related to similar phenomena on rounded bodies, except that in the sharp-edged body the phenomenon is always located at one spot—the edge, whereas in the rounded one it is movable.

To summarize: the force on a body is dependent upon the nature of the flow about it, and the flow is determined by the relation, or ratio, between the pressure and viscous forces, as the first factor, and upon eddies as the second. Hence study must follow these two lines of investigation.

A first line of attack is Euler's "ideal flow"; a second is the Helmholtz-Rayleigh "discontinuous flow." The latter is a modification of the former, made to adjust it more nearly to observation.

Ideal fluid

The "ideal fluid" is an abstraction since it assumes an incompressible, frictionless fluid that is free from rotation. It is the basis for Bernoulli's fundamental formula. It is seen in its most perfect realization in electric and magnetic phenomena. Flow of current through a liquid electrolyte has even been used successfully to solve a few complex cases of airflow, where compressibility is a factor. The flow of stress in a plate bears much resemblance to ideal flow.

To assume air to be incompressible seems at first sight to be extreme. At velocities near that of sound the error is large, but at 175 m.p.h. it is only about 2½ per cent, and at ordinary velocities it is negligible. When studying air flow over high mountains, however, consideration must be given to volume changes. Difference in elevation also is neglected, except for high mountains. The element of friction, however, is all-important in practice; hence we first assume it to be zero and then consider its effect. When viscosity (friction) enters, the fluid can no longer be irrotational.

Upon a rounded body in uniform motion with respect to the fluid, or fixed in a uniformly moving fluid, *the ideal flow can exert no force* whatsoever. Fig. 1 shows the flows in two dimensions about indefinitely long cylinders and flows in three dimensions about spheroidal bodies. At the point or line where the surface is square with the flow direction, which is called the "stagnation point" or "stagnation meridian" the flow must come to rest and then turn 90 deg., to move parallel to the surface. Bernoulli's theorem shows what the external pressure must be at such a point.

Adapting this theorem to the use of the aerodynamic engineer, we obtain

$$\tfrac{1}{2}\rho v_0^2 + p_0 = \tfrac{1}{2}\rho v^2 + p = C,$$

where p_0 and v_0 are the velocity and pressure of the approaching "free stream," v and p those at any given point, and C the total pressure of the fluid "at infinity," i.e. at a great distance from the body. This merely states that the velocity pressure, $\frac{1}{2}\rho v^2$, plus the static pressure, p, at all points along a stream line is constant and equal to that of the free stream at infinity. Since we are interested only in the difference of external pressure, we can say that if the velocity at the stagnation point is zero, then $p_s - p_0 = \frac{1}{2}\rho v_0^2$, or the difference of pressure equals the velocity pressure of the free stream. The stagnation pressure, $\frac{1}{2}\rho v_0^2$, is usually denoted by q, hence $p_s - p_0 = q$. Tests invariably show this equation to be true, except for very high velocities, as for example, those of projectiles. The equation is the basis of the pitot tube. Very viscous fluids are another story.

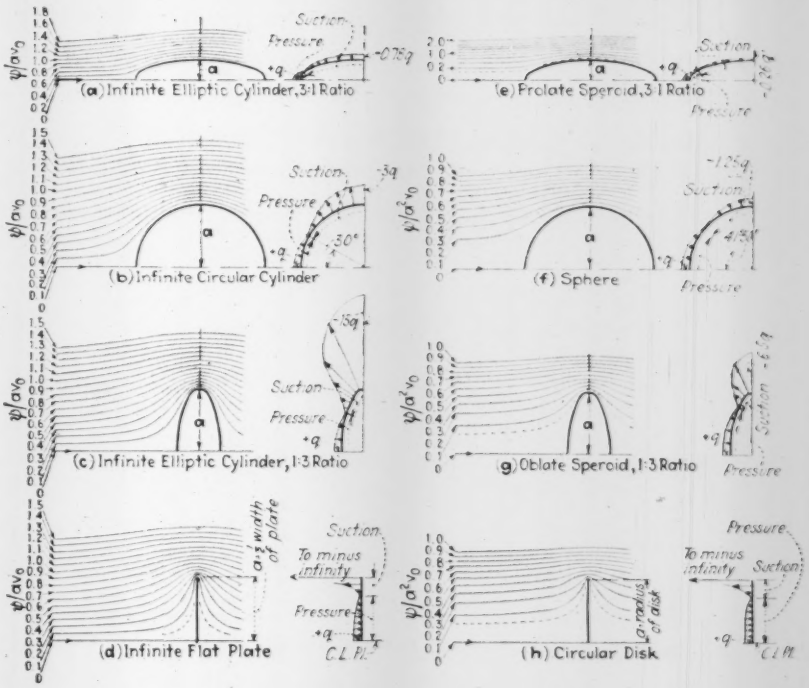
Considering the stagnation point as zero and measuring around the cylinder or sphere from it, the velocity at successive points must increase progressively as more fluid is deflected around the body, so that at 90 deg. we find relatively high velocity. For a circular cylinder the surface velocity at 90 deg. is double that of the free stream, hence the pressure is

$$p_{90} - p_0 = \tfrac{1}{2}\rho v_0^2 - \tfrac{1}{2}\rho\,(2v_0)^2 = -3q,$$

i.e. a *suction* 3 times as great as the stagnation pressure. For a sphere the velocity is 1½ times, and the suction at 90 deg. is 1.25 q. Then from 90 to 180 deg. in the ideal fluid the velocity diminishes symmetrically. Clearly, therefore, there can be no resultant force on the body in an ideal flow. See other bodies in Fig. 1.

Similarly the flow about a thin plate square to the flow direction develops a stagnation point at the center of a square, or along the central meridian of an infinity long rectangle. The outward flow along the plate on each side of this point causes ever greater velocities until the edge is reached, where, according to the ideal theory, the flow turns and becomes symmetrical on the rear, so that there is no resultant force. (Actually the sharp edge upsets the ideal flow in the rear.) Clearly the pressure on the windward face cannot be uniform if Bernoulli's theorem is true. Experiment shows that the actual pressure *is not* uniform, which is contrary to common understanding. An old concept of a pyramid of stagnant air tends to cloud correct understanding.

If the plate is at an angle to the wind less than 90 deg., a greater part of the flow passes over the "trailing edge,"



For infinite cylinders and flat plate the volume of flow per sec. per unit length between two stream lines, ψ_1 and $\psi_2 = \left(\frac{\psi_1}{av_0} \cdot \frac{\psi_2}{av_0}\right) av_0$. For example, between two lines whose $\frac{\psi}{av_0}$ interval is 0.1 the volume of flow is 0.1 av_0 per unit length a=half width of body; v_0=approach velocity, $q = \frac{1}{2}\rho v_0^2$; The scale of all stream lines is the scale of "0". All ideal flows are similar for similar bodies

For bodies of revolution, the volume flowing per second in the annular space between two stream line "surfaces" ψ_1 and $\psi_2 = 2\pi\left(\frac{\psi_1}{a^2v_0} \cdot \frac{\psi_2}{a^2v_0}\right)a^2v_0$ For example, between two surfaces whose $\frac{\psi}{a^2v_0}$ interval is 0.1, the volume of flow is $0.628\,a^2v_0$

FIG. 1—DIAGRAMS of flow of an ideal fluid (frictionless, incompressible and irrotational) around various geometrical bodies, showing location of the pressure and the suction areas.

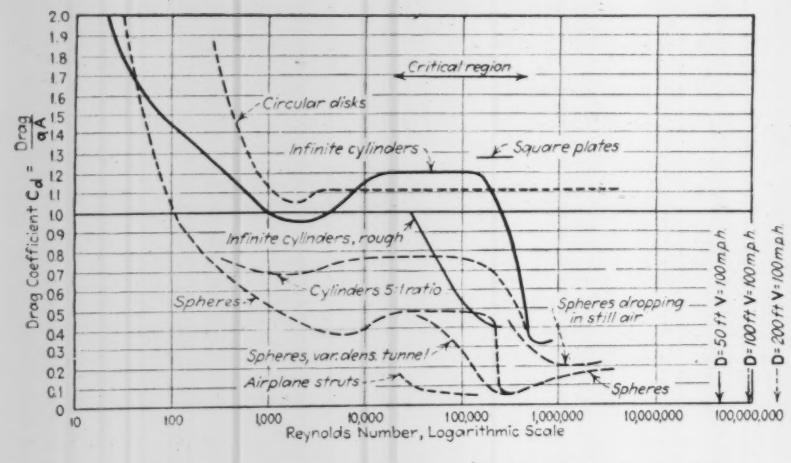


FIG. 2—DRAG or resistance coefficients for long cylinders and spheres. Note the sharp decline of the coefficient in the so-called critical region where the effect of eddies is pronounced.

than over the "leading edge," hence the stagnation point—or line—lies nearer the latter, and the pressure is not symmetrical on the windward face. Assuming for the present a full ideal flow on the rear, it is clear that there is a force couple tending to turn the plate square to the flow direction. Now, although a solid body differs materially from an infinitesimally thin plate in the character of flow about it, nevertheless it also experiences a turning moment, and on a long narrow building such as the Meyer-Kiser Building, which was damaged in the Florida hurricane of 1926, such a moment would cause a material increase in the shearing stress of the columns at one end and a diminution at the other end. This stress has never been considered to my knowledge.

Stokes found, for rounded bodies at very low velocities or in highly viscous fluids, i.e. for values of R less than 1, that the actual flow accords closely with that of the ideal theory.

For high velocities, or for dimensions in the civil engineering range, R has high values, even as great as 100,000,000 or more, and here the flow differs. On a rounded forebody, as of a tank, the agreement is still good, so that the ideal theory gives a good approximation, although eddies largely determine the character of flow. Before considering structures having flat surfaces, it is most important to have a clear understanding of the actual flow about rounded bodies.

Actual flow and eddies

Karman's mathematical study of the eddies that pass in staggered array from a cylinder—the eddies that cause a wire to sing in the wind—and the experimental confirmation of his computed spacing, have led the way to a better understanding of the resistance of bodies, or to use the aerodynamic engineer's term, the "drag" or force parallel to the flow direction. It has already been remarked that, where friction acts, the flow is changed. This friction of course is due to the fact that the fluid (air) at the body surface adheres and is at rest with respect to the body, while that at successively greater distances moves at greater velocities, until finally the approximate velocity of the ideal flow is attained. This layer of ever-increasing velocity has been termed by Prandtl the "boundary layer." For low values of R the relative loss of energy in this layer is small, hence in Stokes' range there is little divergence from the ideal theory. One lamina of fluid slides, or shears, over the next in what is aptly called "laminar flow." Everyone has seen the small ripples on a sheet of water in a wind of low velocity, which soon die away. At higher velocities the waves increase in height as they travel (because the effect is self-generating) until finally they break in white caps. So, by analogy, the boundary layer for higher R becomes turbulent. The energy that had been transferred from pressure to velocity on the forebody is—in the ideal theory—just sufficient to cause the flow to contract again on the afterbody, where is is again transferred to pressure. But when it has in part been dissipated in the boundary layer, it is no longer capable of causing the flow to contract. Naturally, therefore, the boundary layer peels off into two laminar or turbulent sheets.

Of great importance is the fact that the resistance, or drag, of the cylinder for varying values of R is closely related to the eddy frequency of this range. In the region $R = 20{,}000$ to $R = 500{,}000$ for cylinders, many observers have found a rapid decline in the resistance when it is related to stagnation pressure. This region is known as the "critical region," and it forms a rather wide band depending—in detail—upon still another friction factor, i.e. roughness. In aerodynamic methods there is used a term "drag coefficient," C_d, which as a pure number is free from considerations of dimension and velocity. Since total resistance or "drag" consists of (average unit pressure) × (projected area), this term divided by q, the velocity pressure of the free stream, and by the projected area of the body gives the drag coefficient, or

$$C_d = \frac{\text{drag}}{q \times (\text{projected area})}.$$

In Fig. 2 the drag coefficients of an infinitely long—i. e., a two-dimensional cylinder—and of a sphere are given. The rapid decline in C_d at the critical region is clearly shown.

To determine the actual resistance of a 1-ft. length of any given *long* cylinder at any given velocity, first determine R, then find C_d, and multiply C_d by q and by the projected area (1 ft. × D) of the cylinder. For convenience, in air of "standard" density (mass) and temperature, i.e. .002378 slugs (slug = 32.2 lb. per cu.ft.) and 59 deg. F., $R = 9400VD$, where V is in m.p.h., or $R = 6400\,vD$, where v is in ft. per sec., and D is in feet. The value of q at any velocity is $0.002558V^2$, or if more convenient,

$$q = 25.58 \left(\frac{V}{100}\right)^2$$

Above the critical region for cylinders, experiments have not been extensively made. This is unfortunate because chimneys, large stacks and tanks lie in this region. But there are values up to $R = 1{,}000{,}000$, which show a tendency to stabliize at $C_d = 0.35$ for long cylinders. At the critical region Relf found a sudden increase in the frequency of the eddies. Such observations as I have been able to make on the smoke issuing from chimneys (crude though they be) seem to indicate further continuance of the relation between frequency and drag. This is a field that offers opportunity to many engineers of an inquiring nature. To know accurately the pressure on large stacks and tanks, we must have such information.

Converting to civil engineering units, resistance $= 0.35q = 0.35 \times .002558V^2 = .0009V^2$ where V is in m.p.h. This would give 9.0 lb. per sq.ft. at 100 m.p.h. (Note that I do not recommend an assumption of 100 m.p.h. but use it here for simplicity only.)

Extended research has outlined the reason for the change of drag for cylinders and for spheres. Fig. 3, curve I*a*, shows the quatrefoil of pressure for ideal flow about a circular cylinder of infinite length, which is of similar type to that for the flow about spheres. (See Fig. 1 also.) The forward and aft foils are positive, and the larger lateral ones negative. The diagram is obtained by plotting radially outward from any surface point the pressure (or suction) at that point. The pressure intensity at 0 deg. is q, and the suction at 90 deg. is $3q$, as already stated. Since the aft foil is positive and identical with the forward one, there can be no resultant drag on the body for ideal flow. The formula for the pressure intensity is $p/q = 1 -$

$4 \sin^3 \phi$, where ϕ is the angle measured from the stagnation point. Clearly the positive maxima are at 0 deg. and 180 deg., and the negative maximum is at 90 deg. The zero values are at 30 deg. and 150 deg.

In actual flow the forward 30 deg. is substantially unchanged. See Fig. 3, curve II, for the largest-diameter cylinder yet tested, the data for which were furnished the writer by A. Fage, National Physical Laboratory, Teddington, England. The point of zero pressure moves variously to 31 deg. to 38 deg. for differing conditions of test and length of (short) cylinders. Aft of 30—38 deg. the intensity of pressure (suction) falls short of that due to the ideal flow, as shown by curve II for an infinite cylinder at $R = 962{,}000$, curves III*a* and *b* for a short cylinder of length equal to 15 diameters and $R = 168{,}000$ and 336,000, and IV for a short cylinder of length equal to 3.8 diameters. (Models III and IV were actually only of half these lengths, but stood on a platform which I have considered as a plane of symmetry). Note the change between III*a*, which is below the critical region, and III*b* which is above it, also the large increase of the suction with a forward component and the diminution of the suction in the wake in III*b*. The other curves are above the critical region.

I have found that an approximation to the actual flow can be made by modifying the ideal formula into the form $1 - b \sin^2 c\phi$. Curve I*b* is based on $1 - 3.6 \sin^3 \left(\frac{90}{85}\phi\right)$; another form is $1 - 2.9 \sin^2 \left(\frac{9}{8}\phi\right)$. In each case a constant suction aft is assumed, as shown in Fig. 3. The respective drag coefficients of the cylinder from these two forms are 0.363 and 0.359.

By test, in the wake of a sphere and at high values of R, there has actually been found a pressure instead of a suction, but not as great as is required by the ideal theory. Hence the low drag at high values of R. There is some indication that such is true also for circular cylinders at high values of R.

To summarize: it is found that the resistance, or drag, is a mathematical function of R in Stokes' viscous range, in which eddies do not form, but above $R = 1000$ the drag is dependent almost solely upon the eddy formation and the flow pattern about these eddies. At the critical region there comes a sudden change in the sectors where the eddies form, which thereupon move aft and allow a contraction of the outer flow, also a marked change in frequency. This results in less suction in the wake, and in one instance at least it has actually resulted in pressure. Of more importance, however, is the change in the lateral suction, which develops more nearly to the quatrefoil diagram of the ideal fluid and hence becomes more nearly self-equating, so that its effect on the total drag declines.

In passing, I again call attention to the high value of the lateral suction at 70 to 90 deg.—namely, from q to $2.6q$. This has been sufficient in one instance materially to flatten a large steel plate stack. And I also call attention to the intermittent formation of the eddies, which is what makes wires sing in the wind.

The actual flow about elliptical cylinders is better than about circular ones, and the actual flow about a streamline strut such as is used in airplanes is vastly better. The drag of the latter is shown in Fig. 2.

Short cylinders differ materially from *long* ones in that there is a chine at the end which causes effects like those of other sharp-edged bodies. Hence sharp-edged bodies will be considered before discussing short cylinders. It is to be emphasized that all of this discussion deals with the difference of pressure between the body surface and the approaching free stream.

Discontinuous flow

It has been brought out that the change in drag coefficient of geometric bodies is closely related to the point at which the eddies form and to their frequency. In sharp-edged bodies the conditions differ. Just as, when stress lines pass around a sharp re-entrant corner they crowd so close to the corner that the stress intensity may exceed the strength of the material and cause fracture; or, when water passes around a sharp pipe bend there is high turbulence; so when the fluid (air stream) reaches the sharp edge of a plate, where theoretically the velocity is infinite, it fractures. Water issuing from a square-edged orifice, it is well known, shows contraction of the jet. The flow of air colored with smoke past a prismatic block in a wind tunnel shows the same characteristic.

Lord Rayleigh undertook to apply this observation as a corrective to the ideal flow theory by assuming a "surface of discontinuity." In the wake he assumed a pressure equal to that of the free stream, which for air is not a very good assumption. Outside of this surface he assumed an ideal flow. In an attempt further to correct his flow, I have assumed that there is suction of a given amount in the wake, whereby a correction factor is obtained. I have correlated the three flows by application to a given infinite flat plate at 90 deg. to the wind direction, in Fig. 4. There are shown, for the windward face, the ideal flow pressures derived from Bernoulli's formula; next, the Rayleigh pressures; then actual pitot tube readings; and finally the Rayleigh pressures modified as just mentioned, by assuming a uniform suction of $1.38q$ in the wake. This adjustment gives reasonable agreement with test. For more perfect agreement

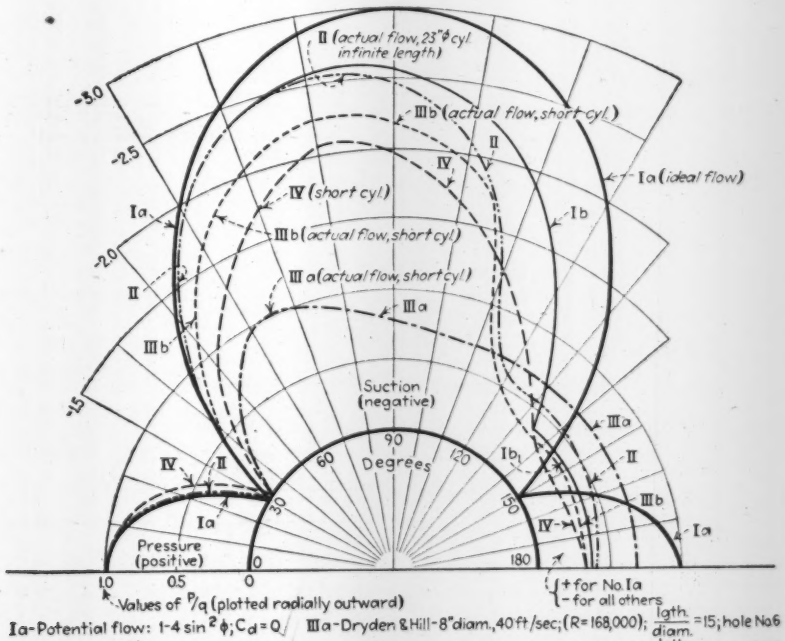

II (actual flow, 23"ϕ cyl. infinite length)
IIIb (actual flow, short cyl.)
IV (short cyl.)
Ia (ideal flow)
IIIb (actual flow, short cyl.)
IIIa (actual flow, short cyl.)
Suction (negative)
Pressure (positive)
Degrees
-.50
-.25
-2.0
-.15
30
60
90
120
150
180
1.0
0.5
0
Values of P/q (plotted radially outward)
+ for No. Ia
- for all others
Ia-Potential flow: 1-4 sin² ϕ; Cd = 0
Ib-1-3.6 sin² (90/85 ϕ) Cd = 0.363
II- Fage 23"diam., 80 ft/sec. (R = 962,000) infinite length
IIIa-Dryden & Hill-8"diam., 40 ft/sec; (R = 168,000); lgth./diam. = 15; hole No 6
IIIb-Dryden & Hill-8"diam., 80 ft/sec; (R = 336,000); lgth./diam. = 15; hole No 6
IV- Prandtl & Betz; gasholder model, hole No. 2, 11.8 diam.
98 ft/sec. (R = 605,000) lgth./diam. = 3.8

FIG. 3—DIAGRAM showing positive and negative pressure areas on various cylinders. Particularly notable is the high lateral suction at angle of 50 to 70 deg. Tabular matter gives equations for pressure intensity (p/q) and velocities for the various cylinders and for various Reynolds' numbers.

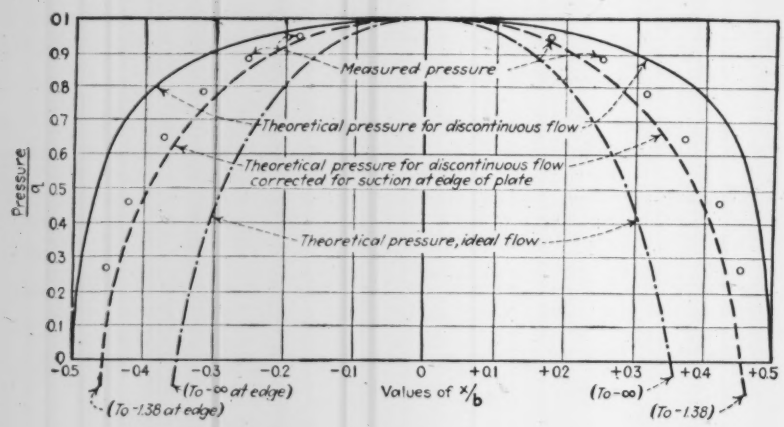

Measured pressure
Theoretical pressure for discontinuous flow
Theoretical pressure for discontinuous flow corrected for suction at edge of plate
Theoretical pressure, ideal flow
Pressure q
Values of x/b
(To -∞ at edge)
(To -1.38 at edge)
(To -∞)
(To -1.38)

FIG. 4—PRESSURE INTENSITY on the face of an infinite flat plate set 90 deg. to the wind. By assuming suction at the edge of the plate (or in the wake of the air stream) a curve is obtained midway between the ideal and discontinuous flow curves that corresponds closely with actual pitot-tube readings.

it would be necessary to know the thickness of the boundary layer on the windward face and to make correction for it. Such correction for boundary layer, when applied to the test results from several airship models, has shown remarkably good results, but the data are not at hand to make it for plates, nor does it seem necessary for use by the civil engineer.

Actual flow about flat plates

Eiffel was among the first to make wind-tunnel tests on flat plates. Duchemin was perhaps the first to make any scientific tests at all. For 100 years sporadic attempts at wind tests have been made by practicing engineers who were faced with specific problems in design, as for instance the Forth Bridge. Some used small plates, some large ones. Three questions arise: (1) Is size a factor? (2) Is shape a factor? and (3) Is velocity a factor? Before answering these questions it may be enlightening to consider some test results.

Circular Disks—While studying smoke-ring formation behind small disks, Simmons found a drag coefficient of 1.84 at $R = 282$. At 20 to 80 m.p.h., Eiffel found a coefficient of 1.06 to 1.08 for 6- and 12-in. disks. Over a wide range of R, and for disks from $1\frac{1}{4}$ up to 16 in., Wieselsberger found coefficients of 1.07 to 1.13, and the National Advisory Committee for Aeronautics for a 5.9-in. disk in the range $R =$ 210,000 to 4,440,000, found coefficients of 1.08 to 1.14. In determining Reynolds' number the diameter of the disk is used. For the civil engineer a coefficient of 1.11 is suitable. (See Fig. 2). Converting this, resistance $= 1.11q = 1.11 \times 0.002558V^2 = 28.4$ lb. per sq.ft. at 100 m.p.h., *again using this velocity only for convenience.*

Square Plates—Eiffel tested plates ranging from 4 to 40 in. (about) at velocities of 20 to 80 m.p.h. His coefficients, regardless of velocity, ranged as follows: 4-in. plate, 1.04; 9.8-in., 1.07; 20-in., 1.19; 28-in., 1.23; 40-in., 1.26. He judged that the coefficient for large plates would become fixed at 1.28. (See Fig. 2). For large plates, 5x5 ft., 10x5 ft., and 10x10 ft., at 0 to 25 m.p.h. Stanton found coefficients of 1.24 and 1.26. Converting this, resistance $= 1.28 \times 0.002558V^2$, or for convenience 33 lb. per sq.ft. at 100 m.p.h.

The Weather Bureau formerly used $0.005V^2$, based on Smeaton's observations. Later it changed to $0.004V^2$. The latest basis is $0.0033V^2$, which agrees with the result just obtained.

Rectangular Plates—A square plate or circular disk permits flow about its entire periphery, i.e. three-dimensional flow. In an infinitely long rectangle the flow takes place only about two edges, i.e. two-dimensional flow. Hence the coefficient must differ materially with the ratio of the lengths of the long and short sides. In a set of tests of small plates Wieselsberger found a coefficient of 1.11 for squares; a rapid increase to 1.20 for a length/width ratio of 5; and 2.0 for infinitely long plates. Had the plates been larger, the range would likely have been from 1.28 to 2.0, since Stanton's tests of 5x5-ft., 10x5-ft. and 10x10-ft. plates showed 1.24 to 1.26 for ratios of 1 and 2. Normal pressure tests of structural shapes having widths of $2\frac{1}{2}$ to $7\frac{1}{2}$ in. show a coefficient of substantially 2.0 so that this figure can be accepted as correct for long narrow plates. For this coefficient, resistance $= 2.0 \times 0.002558V^2 = 0.005116V^2$, or for convenience 51.2 lb. per sq. ft. at 100 m.p.h.

It must be remembered that all of these tests apply only to thin plates and not to buildings, which differ materially. Moreover, they relate to exposed members, signboards, etc., which are free-standing and not shielded by other similar members.

The three questions therefore can be answered as follows:

(1). Size *is* a factor. Small square plates show lower resistance than large ones, hence a test made with a small plate exposed on a bridge will not give the pressure correctly for a large building or other exposed surface.

(2). Shape *is* a factor, in that long narrow rectangles and structural shapes show a higher coefficient than short rectangles or squares and disks.

(3). Velocity *is* a factor, in that the pressure varies with the velocity squared. If only the coefficient be considered, however, velocity has no known effect in the civil engineering range.

It is a fact of importance that there is *no change in drag coefficient with velocity*. The sharp corners cause surfaces of discontinuity even at low velocities, and no movement can take place in these generating origins of the eddies. Therefore, there is no reason to expect a change of the drag coefficient with velocity for flat plates. As a matter of fact, this is also largely the case for solid bodies, such as buildings.

Pressure distribution on flat plates

Mention has already been made of the fact that if Bernoulli's formula is true, the pressure on a plate cannot be uniform. Fig. 4 illustrates this fact, and tests confirm it, even recalling that discontinuous and not ideal flow obtains. Tests on building models also confirm it.

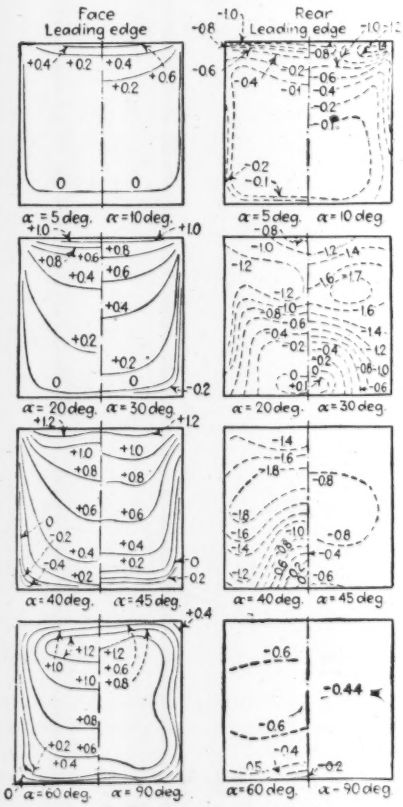


FIG. 5—PRESSURE CONTOURS on a square plate, in lb. per sq.ft., at a velocity of 32.8 ft. per second for wind at various angles. Plate was 20x20 in. except for 90 deg., where a 10x10-in. plate was used.

Probably reference to tests made by Eiffel will illustrate best. Fig. 5 shows a square plate about 20x20 in. (except that the data at 90 deg. were obtained for a 10x10-in. plate) at varying angles to the wind. For angle, $\alpha = 90$ deg., the plate was square to the wind; for $\alpha = 0$ deg., parallel to it. The contours were derived for a velocity of 32.8 ft. per sec., for which the stagnation pressure is 1.28 lb. per sq. ft., and the contours represent multiples of 0.2 lb. per sq. ft. The stagnation pressure would be represented by a contour 1.28, but the highest contour actually shown is 1.2. Due to symmetry the contours are mapped on only one-half the area of the plate for each angle of attack. One set of diagrams deals with pressure on the windward side, the other with suction on the leeward side. The effect of the three-dimensional flow about the four edges is clearly shown. Near the edges, of course, the contours are uncertain, because the pressure holes from which the pressures were obtained could not be located at the exact edge. The higher pressure and suction values are generally toward the leading edge. At $\alpha = 90$ deg. there is almost perfect symmetry of pressure on the windward face, with a rounding of the contours at the corners, and the suction is so nearly uniform that no contours are drawn.

For rectangular plates much the same comments obtain, except that the pressures and suctions are somewhat higher, because the flow is largely two-dimensional.

Inclination effect

It is interesting to compare Duchemin's formula with the results of more recent tests. Fig. 6 shows Eiffel's tests of the coefficient of normal pressure c_n, (i.e. the pressure normal to the plate divided by $q\,a\,b$) for square and rectangular plates, as compared with Duchemin's formula, for angles from 0 to 90 deg. The ratio of the coefficient for any angle to the coefficient at 90 deg. is the ordinate, and the angle α is the abscissa. The curves shown are the results from eight plates for which the length/width ratio, a/b, varies from 9 to $\frac{1}{6}$. The ratio 1 means a square plate. The curves for 9 and 6 show some resemblance to Duchemin's formula, but differ in shape; all the others show a remarkable peculiarity in the form of a peak which is most pronounced for the square plate. Clearly, no single formula can hold for all shapes of plate.

Turning moment on a flat plate

It was mentioned previously that there is a turning moment on a flat plate at angles α less than 90 deg. Most indicative of this moment is the movement of the center of pressure from the center, expressed as a ratio of the width, b. For 90 deg. the center of pressure is at the center line in all cases. For other angles the movement depends upon the length-width ratio, i.e. the shape. The extreme position, for $\alpha = 0$ deg. is at the quarter-point.

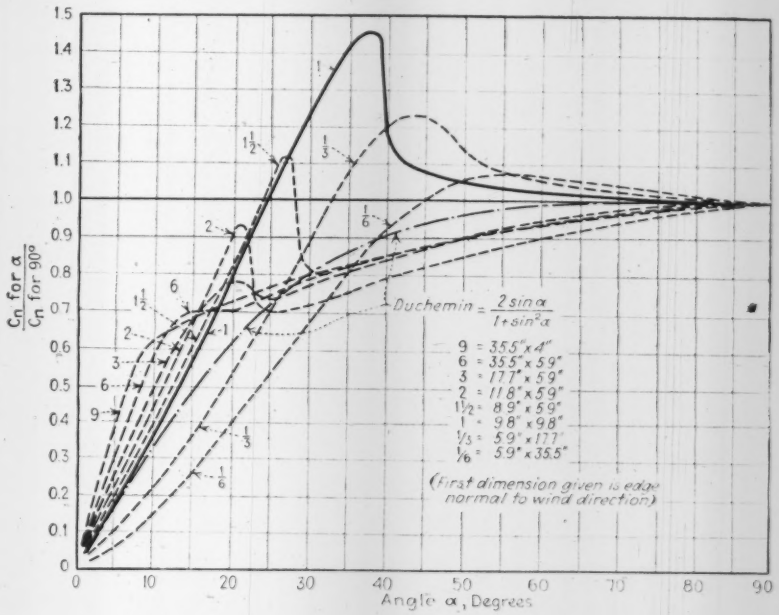

Cn for α / Cn for 90°
Angle α, Degrees
Duchemin = 2 sin α / 1+sin²α
9 = 35.5" x 4"
6 = 35.5" x 5.9"
3 = 17.7" x 5.9"
2 = 11.8" x 5.9"
1½ = 8.9" x 5.9"
1 = 9.8" x 9.8"
⅓ = 5.9" x 17.7"
⅙ = 5.9" x 35.5"
(First dimension given is edge normal to wind direction)

FIG. 6—EFFECT on normal pressure coefficient of inclination of plates to the wind. Curves compare Duchemin's formula with tests by Eiffel on plates of length-width ratios from 1/6 to 9.

Short cylinders

Space does not permit of complete discussion of the effect of the chine at the end of a short cylinder, but it causes eddy formation just as does any other sharp edge. The effect can be clearly seen in the smoke that issues from a high stack when the wind is blowing gently. Just as a short rectangle shows a lower drag coefficient than a long one, so does a short cylinder show less than a long one. The difference depends upon the ratio of length to diameter. Wieselsberger made an extended series of tests for cylinders for which this ratio was 5. These are shown in Fig. 2. An idea of the change in pressure distribution may be gained by a study of the curves of Fig. 3.

This paper has attempted to introduce briefly the subject of wind to the civil engineer. The discussion is also true of water, since Reynold's number contains both the density and the viscosity and therefore is applicable either to air or to water. This is one marked advantage in the use of dimensionless coefficients as shown by the work already done in aerodynamic science. Geometric bodies were used to introduce the subject because they throw much light on the essential fact that the character of the flow determines the pressure on the body. Some facts dealing with plates were developed which obtain regardless of velocity. These are translated into the language of the civil engineer, and the resistance is related to the velocity. It should be repeated that the use of 100 miles per hour was an illustration only and must not be considered as the writer's opinion of the velocity to be used in practice.

Longest Pipe Line of 1933 Built in Arizona Desert Country

THE 217-mile, 10-in., natural-gas line of the Western Gas Co., running from Douglas, Ariz., through Tucson to Phoenix, is believed to be the longest pipe line built in 1933.

As it is located in semi-desert country, its construction was attended by numerous difficulties such as crossing washes and gullies, clearing scrub brush and cactus and ditch digging in hard materials. According to the Lincoln Electric Co., whose shielded arc-welding process was used in laying the line, the pipe was supplied in 40-ft. lengths with 30-deg. beveled ends. Backing-up rings were used to facilitate welding, which was done as the pipe was rotated on dollies, using a chain wrench. Two beads of weld metal were applied. The line was tested under 650 lb. per sq.in. gas pressure. An average of 55 joints per man per day was maintained by the welders of the organization of H. C. Price, Bartlesville, Okla., the contractor.

Surface-Track Construction With Vibrated Concrete

Track of new design to insure rail alignment and prevent joint failures is constructed on 11,900 ft. of experimental surface-line railway on Brooklyn, N. Y., street

By H. J. Kolb
Chief Engineer, Way and Structure, Brooklyn & Queens Transit Corp., N. Y.

TWO TYPES of concrete-embedded surface-track structures designed to reduce maintenance costs have recently been completed on Bay Ridge Ave., in the Borough of Brooklyn, New York City. The objective of the design was to provide a thoroughly rigid structure in which the track and pavement would act as a unit. Certain design variations were introduced so that any differences in performance under operation might be observed. The double-track work and paving were constructed half at a time to facilitate the operation of trolleys and vehicular traffic and also to simplify the construction methods.

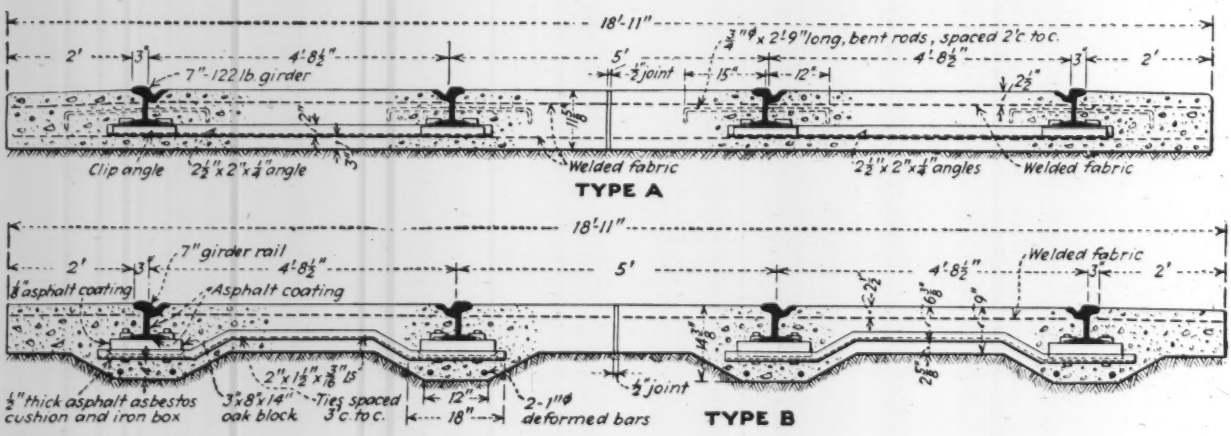
Two design types

Type A construction (Fig. 1) consists of 8,690 ft. of track. This design provided for the use of 2½x2x¼-in. angle spacer bars every 6 ft. The 7-in., 122-lb., grooved girder rails were fastened to the spacer bars by clips. Tiebars between the rails were omitted, but additional rail anchorage was obtained by ¾-in. round bars, 2¾ ft. long with right-angle bends at each end, inserted through holes spaced 2 ft. apart in the rail web. The concrete pavement was reinforced below the ties 2½ in. from the bottom of the slab and also 2½ in. below the surface with welded No. 6 gage steel fabric, the members being spaced 6 in. both ways. The pavement had a uniform thickness of approximately 11⅝ in.

Type B construction consists of 3,010 ft. of track. Truss-shaped cross-ties consisting of two 2x1½x⅜-in. angles set 8⅛ in. apart were spaced 3 ft. c. to c. The rails were fastened to the cross-ties by clips. No rail anchorage bars, as used in type A construction, or any other tiebars, were used. Steel-fabric reinforcement similar to that used in type A construction was placed 2½ in. from the top of the pavement only. The bottom of the slab was shaped to follow the general contour of the cross-ties, as shown in Fig. 1. The rails were coated with bituminous material to prevent bond with the concrete.

It will be observed that the rails in the type A section were bonded and anchored into the concrete in an effort to make both rail and concrete structures act as a unit, while in type B construction an effort was made to insulate the rails from the concrete pavement structure so that they would act as two independent units, the idea being that in the latter case vibrations of the rail would be dissipated through the rail and bituminous coating in such a way that they would not be transmitted to the concrete structure, while in the former case it was intended that the rail vibrations would be transmitted to the concrete structure and there dissipated. No bituminous coating was applied to the rails in type A construction, but every effort was made to secure as close a bond between the rails and concrete as possible.

FIG. 1—TWO TYPES of experimental surface-track construction employed on street railway in Brooklyn, N. Y.



General track construction

The grading operations consisted in the removal of old track and the careful preparation of a new subgrade with the addition of filling material where required and the consolidation of the whole area by the use of an 8-ton roller.

In the case of type B construction the irregular contour of the subgrade was excavated by hand after the rolling. As type A design provided for steel-fabric reinforcement below the spacer bars, it was necessary to install this reinforcing as the first step in the track construction. Rails and spacer bars were then installed in the usual manner, sections of the reinforcement being cut out around the area occupied by the track supports, which consisted of precast concrete units. The track work was then brought to proper grade and alignment, the mesh reinforcement being pulled up into position when the concrete was placed.

All rail joints were thermit-welded before the concrete was placed. Immediately after the welding process the joints were ground with a swing-frame grinding machine and left in this condition until the day before traffic was permitted on each section, at which time a grinding car with two reciprocating rail grinders was operated over the entire length of the section that was to be opened for traffic. The grinding took care not only of the joints but of the entire rail length; although the rail was new, it was ground from one end of the work to the other to eliminate any irregularities in its surface.

Placing the concrete

Concrete for the track pavement was transported from a central proportioning plant in truck mixers carrying 5 cu.yd. per truck. The average length of haul from plant to job was 4½ miles, and the average length of time between loading at the plant and discharging at the job was about 25 min. The specifications provided for a minimum compressive

FIG. 2—CONCRETE chuted into track direct from truck mixer and spread by hand for compaction by traveling vibrator.

strength, at 28 days, of 3,000 lb. per sq.in. The average actual strength obtained, as determined from the tests, was 4,200 lb. per sq.in. The concrete mix was about 1:2:3½. All ingredients except the water were measured by weight. Due to the amount of structural and reinforcing steel in the track section, it was found desirable to use a grading of coarse aggregate that would produce concrete of the desired workability and assure its proper placement around the steel. The relative proportions of various stone sizes were as follows: 20 per cent of ¾-in., 50 per cent of ⅝-in. and 30 per cent of ⅜-in. Trap-rock aggregate was used.

Concrete of stiff consistency varying between 4.7 and 5.25 gal. of water per sack of cement, depending on weather conditions, was discharged from the truck mixers (Fig. 2) through a portable chute to position in the track area. The concrete was then evenly distributed by hand over the area beneath the rails, and the lower reinforcement was pulled up into place by means of hooks as the work progressed. This portion of the concrete was then vibrated by means of the device shown by Fig. 3. This equipment, riding the rails and electrically operated, passed over the section once at this time.

The concrete was then deposited over the area between the bases of the rails to the elevation to be occupied by the upper layer of steel-fabric reinforcement. After the concrete was spread by hand to proper elevation, the ¾-in. bent-web anchor bars were placed, the steel-fabric reinforcement was installed 2½ in. from finished grade, and then the remainder of concrete was spread, completing the operation. As soon as the concrete was properly shaped, the vibrating equipment was again passed over the area so that complete consolidation might be obtained. The vibrators produced a much denser concrete than would have been obtained with ordinary tamping, the vibratory action forcing all air pockets out of the concrete and insuring a close bond between the concrete and the embedded track structure.

FIG. 3—VIBRATING MACHINE traveling on rails shown in distant and close-up positions. Following vibration, the surface was hand-floated.

When concrete was placed on the second half of the track area, a protecting angle strip was placed over the premolded center-joint material to maintain the proper position of the expansion-joint material.

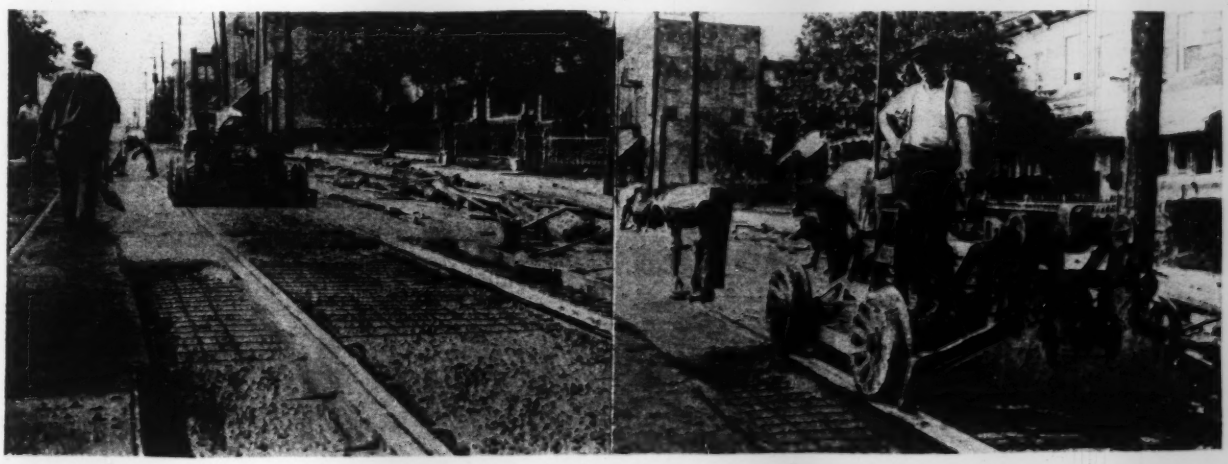
The pavement cross-section between rails and in the dummy strip between tracks is level, while that in the outside track liner strips is slightly pitched toward the roadway pavement. It was necessary, due to the shape of railheads, that the surface of the concrete should be held about ⅝ in. below the top of the rail as indicated. No difficulty was experienced in finishing the concrete by hand-floating methods to the proper levels. As soon as possible after the pavement surface was finished, the entire area was covered with hay kept saturated from ten to fourteen days, depending upon weather and temperature conditions.

The only portion of the street torn up at one time consisted of the width occupied by one track, which was 10 to 12 ft. Under these conditions it was possible to keep the street open at all times to local traffic as well as trolley traffic. The paved track was completed in sections about 1,200 ft. long, and rail traffic was allowed to use the completed sections after the curing period. Construction operations started on July 26 and were completed Nov. 4, 1933.

General discussion

It is felt that the type of construction used on this project will prevent rails from getting out of line and should eliminate to a great extent joint failures; therefore the track and pavement maintenance costs will be materially reduced from those of the usual types of construction

It is believed that savings in track-maintenance costs will be due partly to a decrease in the amount of rail corrugation and particularly to the fact that the entire length of rails throughout the project were carefully ground to a uniform even surface prior to the operation of cars.



No transverse expansion joints were used, and it is the belief that if rails are firmly held in the concrete as they should be, expansion joints cannot function without deforming the rails Only vertical construction joints were used at the end of each day's work. A ½-in. expansion joint was constructed longitudinally between the two track sections in order that these sections might act independently of each other.

The special design and construction features may be summarized as follows: (1) reinforcement is used below the track work as well as in the upper surface of the pavement; (2) special rail anchorage bars are installed through the rail web; (3) there are two special types of track section; (4) all transverse expansion joints are omitted; and (5) all rails are ground before receiving traffic.

The construction work was under the direct supervision of C. W. Burke, assistant chief engineer of way and structure. The contractor for the work was Edwards & Flood, Inc., Brooklyn, N. Y. Truck-mixed concrete was furnished under contract by the Transit Mix Concrete Corp., New York City.

Bridging the St. Lawrence at the Ile d'Orleans

New highway crossing in province of Quebec includes a 1,059-ft. suspension span of prestressed-cable-strand type and off-shore anchorages involving unique design and special construction planning

SOME 7 miles below the city of Quebec and about 15 miles below the famous cantilever railway bridge over the St. Lawrence River, there is under construction a new highway crossing of this stream, or more correctly of a back channel of it. Involving a 1,059-ft. suspension span, deep and difficult foundation work and long approaches, the Ile d'Orleans Bridge, as it is called, has substantial technical significance. It is interesting in a somewhat broader sense in that it is being built as an unemployment-relief measure, construction being spread over four years, from 1932 to 1935, inclusive. The bridge connects the mainland at St. Gregoire de Montmorency with the Ile d'Orleans, lying in the center of the St. Lawrence River. This island, attractive in topography and of considerable historical interest, but hitherto isolated and occupied only by small farms, is thus made easily accessible to Quebec City and opened for real estate development.

The bridge, as shown on Fig. 1, crosses the north channel of the river. Although a large volume of shipping utilizes this channel, the wider south channel is the route taken by the larger ocean-going vessels. Both sides of the north channel are flanked by wide bands of shallow water, necessitating long approach viaducts. On the island side the bank rises quite steeply, and inasmuch as the principal town, St. Petronille, is on the south channel, a new approach road some 2 miles long is required to circumvent this bluff.

As shown in Fig. 2, each of the approaches comprises a series of nine 60-ft. reinforced-concrete girder spans followed by about 1,200 ft. of 150-ft.

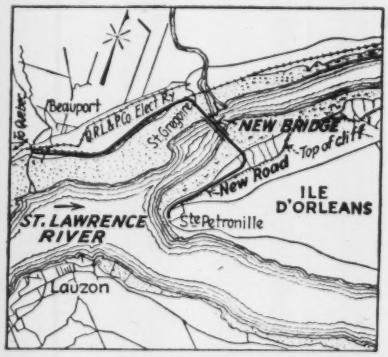


FIG. 1—LOCATION MAP of new highway bridge over north channel of St. Lawrence River connecting Ile d'Orleans with mainland near Quebec.

steel deck truss spans. These approaches are complete, having been built during 1932 and 1933 by the provincial department of public works. In general neither their design nor construction were unusual or notable. Piers are founded on timber piles cut off at about El. 70, and extend up to several feet above high water at El. 92, where they are surmounted by columns carrying the girders. On the concrete-span sections these columns are more in the nature of walls pierced by openings, an expedient adopted largely for the sake of appearance. On the steel-truss sections, a concrete column is used beneath each truss, braced apart by heavy struts. The bridge deck provides a 20-ft. roadway and two 5-ft. sidewalks.

Principal technical interest in the structure attaches to the main channel crossing. This channel, about 1,000 ft. wide, was required to have at least a 600-ft. shipping opening in which the bridge clearance was 106 ft. Studies of various designs from simple trusses to cantilevers and suspension spans led to the adoption of the latter as the least costly and most sightly for the prominent location. Water depths and foundation conditions dictated a main span between piers of 1,059 ft. The remainder of the main bridge consists of 417½-ft. side spans with loaded backstays and 238-ft. deck-truss spans between the anchorages and the cable-bent piers (Fig. 2). Cables are of the prestressed bridge-strand type, towers are of fixed-base flexible design, and a 50-lb. floor slab of tee grids filled with concrete is to be used. Of the foundations, all of which are in reasonably deep water, the north anchorage was the most difficult and represents an ingenious design to provide stability and a construction sequence planned to speed up the work and eliminate troubles with ice or floods.

Foundation conditions are extremely variable. At the anchorage and cable-bent pier on the south end, rock (a limestone-shale) comprises the river bottom, but just beyond the cable-bent pier it dips sharply toward the center of the stream. Some 200 ft. farther out, at the south main pier, the dip of the rock is toward shore; its depth varies from 10 to 30 ft. below mud line and 40 to 60 ft. below low water. At the three north foundations borings to 100 ft. below water failed to strike rock. However, satisfactory material in the form of sand and gravel with some clay was encountered, so that the main pier could be founded at El. 27 and the anchorage at about El. 60, respectfully 65 and 32 ft. below mean high-water level.

The two main piers will be built with compressed-air caissons. The other foundations on the north side will be constructed inside steel sheetpile cofferdams, while those on the south will be built in timber cofferdams. The caissons are now under construction at Quebec, and as soon as the ice in the river breaks up they will be floated to the site. Cofferdam construction for the north piers will start late in April, followed in July by the cofferdams for the shallower south foundations. All piers are to be faced with granite ashlar up to high-water level.

Special anchorage design

Principal technical interest centers on the anchorages which were designed with the erection requirements of both substructure and superstructure contractors in mind. These anchorages are of gravity type and, as such, required particular attention to stability and a reasonable toe pressure. The latter requirement was aided by leaving as large a cavity in the front part of the anchorage as possible, and by placing the pier shaft that is carried by the anchorage 8½ ft. back of the transverse center line. Calculations for the north anchorage show a toe pressure of 5,075 lb. and a

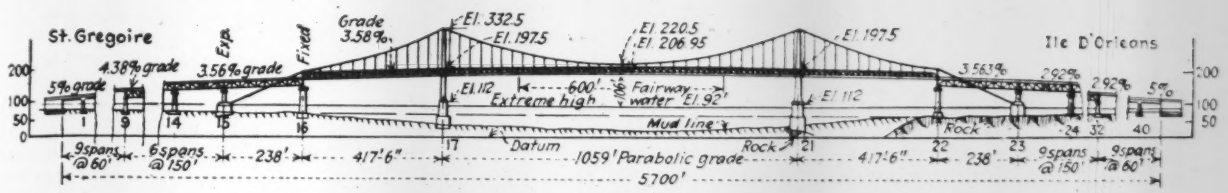


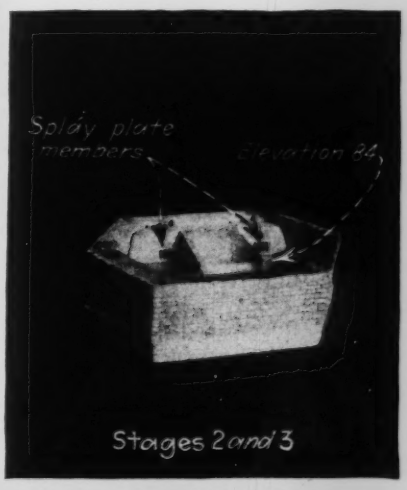
FIG. 2—ILE D'ORLEANS BRIDGE over the St. Lawrence River at Quebec. Spans of long approaches are completed, and work is beginning on the suspension-bridge foundations and fabrication.

heel pressure of 4,285 lb. per sq.ft. with this arrangement, considering water at low stage, approach spans loaded and maximum cable pull. Sliding resistance was also a factor, particularly on this north anchorage (Fig. 4) where rock is not available; the design takes care of it by providing for 70 steel H-section piles to be driven on a batter of 1:2.75. On the south anchorage, it was possible to key the foundation into the rock.

The anchorage design also contemplates a sequence of construction operations looking toward both speed and safety. Briefly, the plan is to drive the cofferdam inclosure, excavate to foundation level and pour a 10-ft. footing course, in the dry if possible. Following this, a wall about 5 ft. thick will be built up to slightly above water level; the stone facing will comprise the outside form for this wall. Concrete for this footing course and masonry backing will be made with a maximum of 4½ gal. of water per sack; it will also include an integral waterproofing compound and will be vibrated. Completion of this first stage thus will provide a watertight cofferdam or box of permanent structure within which the steel contractor can place his anchorage steel and the foundation contractor can complete his concreting in safety regardless of river conditions.

The second stage consists of pouring a block of concrete across the front half of the anchorage up to El. 84.0, following which the anchorage grillage and splayed plate members will be set. In the third stage of the work concrete will be poured and tamped around the anchorage steel, and a block across the whole rear half of the box will thus be added; in addition, the base of the pier shaft, situated between the clear of the two sets of anchorage steel, will be poured. Construction of the upper pier shaft can then proceed as stage 4. The shaft is somewhat wider in its upper portion than it will be in the temporary base constructed for its initial support, but ample provision for stability has been made in proportioning these masses. This pier arrangement is one of the principal features of the design and was worked out so that the approach span to the anchorage can be erected at the earliest practicable moment, as with this span in place the superstructure contractors will have direct access to the work without recourse to floating equipment. Included in stage 4 are also two blocks of concrete filling the up and downstream ends of the box to the level of the stone facing, but still leaving pockets in front of the cable splay plates (see Fig. 3).

After the cable strands are erected and adjusted to length, a fifth concreting stage, comprising all the remaining parts except the breast wall and slab roof over the forward part of the an-



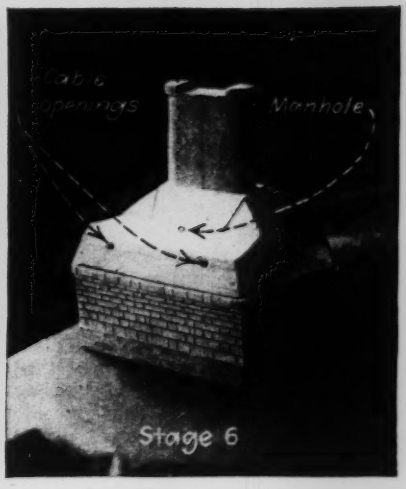


FIG. 3—CONSTRUCTION STAGES for anchorages as shown by balsa wood model. Construction plan based on two main requirements: to provide a watertight cofferdam of permanent structure at the earliest practicable moment (stage 1), and to build the pier as soon thereafter as possible (stage 4), so that the approach span can be placed for early use of the cable erection contractor.

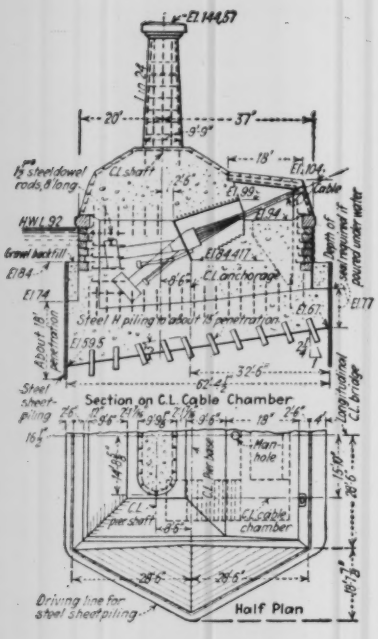


FIG. 4—NORTH ANCHORAGE, located offshore in a 20-ft. depth of water, is founded on clay and gravel. Sliding and overturning tendencies are resisted by steel H-section piles driven on a batter and by concentrating the weight of the anchorage block and the superimposed pier back of the transverse center line.

chorage, can be undertaken. The final concreting (stage 6) can only be done after the cables are loaded and wrapped. Access for inspection is provided, and the inclosing of the splaying cable strands in an asphalt mastic is under contemplation as a definite protection against corrosion. To aid both themselves and the contractors in their planning, the engineers constructed a balsa wood model of the anchorage pier in which the various concrete pours are represented by separate pieces of wood, and five photographs of this model are illustrated in Fig. 3, as indicative of the various stages of construction.

Superstructure

The superstructure of the suspension span is notable for several special details and for the use of prestressed bridge-strand cables and a light-weight floor. Erection is scheduled to be actively under way during 1935.

The towers consist of two legs 218 ft. high above the masonry with X-braced panels, detailed with special regard to appearance, in view of the scenic properties of the site. The main material is of silicon steel, while the bracing and details are of structural carbon steel. In cross-section each leg consists of a plate and angle box 3 ft.x3 ft. 3½ in. in plan, acting as a constant core, but furnished with plate and angle wings, also of box form (Fig. 5). These wings give the tower its characteristic shape, as viewed from up or downstream, are 2 ft. wide and vary in depth from 4 ft. 1 in. at the base to 1 ft. 3 in. at the top of the tower. This arrangement provides a steel section of 340¾ sq.in. at the base of the tower to carry a direct vertical load of 2,678 kips from dead load, normal live load and temperature, the combination being 3,790 kips when wind is included. The co-existing longitudinal bending moment in the tower leg amounts to 4,385 ft.-kips due to the horizontal pull of the cables and the eccentricity resulting from deflection. Unit stresses in compression at any section are specified not to exceed:

$$22{,}000 - \frac{320 \times l_1 \times l_2}{l \times r}$$

where l_1 is the distance of the section from the top of the tower, l_2 the distance from the base of the tower, $l = l_1 + l_2$, and r is the radius of gyration at the section considered, all these dimensions being in inches. This specified unit stress is for the first combination, and a 15 per cent addition is permitted for the second combination (which includes wind), and also for a third combination defined as dead load, congested live load and temperature. Under these stipulations the first case governs throughout the design.

An unusual feature is the introduction of welded structural-steel plates for the built-up shoes. Each shoe will consist of a 1½-in. base plate about 8 ft. wide and 17½ ft. long, upon which are erected 1½-in. and 1¼-in. vertical stiffening plates or ribs, supporting in turn a 1¼-in. top plate, the whole shoe being 2 ft. deep. The area of the ribs in bearing is about 1,350 sq.in., and the maximum calculated compression in any rib is limited to about 7,000 lb. per sq.in. There will be no further base plate on the tower legs, but the vertical angles and plates as well as the heavy base angles will be machined to bear directly on the planed surface of the welded structural shoe.

Cables—The cables are made up of thirty-seven 1⅛-in.-diameter prestressed strands, providing a metallic area of 41.4 sq.in. and, with cedar-wood fillers and 9-gage wire wrapping, yielding a finished diameter of about 10 in. Maximum cable tension is 3,088 kips. A 75,000 lb. per sq.in. unit stress was used in the cable design.

Conventional cast-steel tower saddles and cable clamps are used. Anchorage connections are of button type fitting into special grooved anchor plates developed by the contractor. Suspenders are of double-rope type using 1⅛-in.-diameter prestressed bridge strands connected to the stiffening trusses at every second panel point.

Stiffening Trusses and Floor—The stiffening trusses are of modified Warren type with 16-ft. floor panels, using carbon steel of 60,000-70,000-lb. ultimate strength. Chords are 13 ft. c. to c. with plate-and-angle make-up. The trusses are spaced 31 ft. 8 in. on centers, providing for a 20-ft. roadway and two 5-ft. sidewalks.

The trusses are proportioned for moments and shears and for discontinuous live loading. The intensity of live load is lighter than that usually adopted since the bridge is not likely to be called upon to carry very heavy traffic. On the other hand, the floor system and the local members of the trusses are made adequate at conservative unit stresses for 15-ton trucks with 30 per cent impact.

Floor beams consist of 36-in. plate girders and support 13-in. 35-lb. British standard stringer beams. At the main piers the stiffening trusses are pinned to rocker posts, which are seated on a cross-girder, and the wind load is transferred from the floor to the tower by means of a special detail forming part of this cross-girder and indicated on Fig. 5.

Finger expansion joints at the main towers and at the ends of the side spans will be specially designed to meet the very severe conditions presented by the extreme temperatures of the locality and the sleigh traffic; by making the fingers

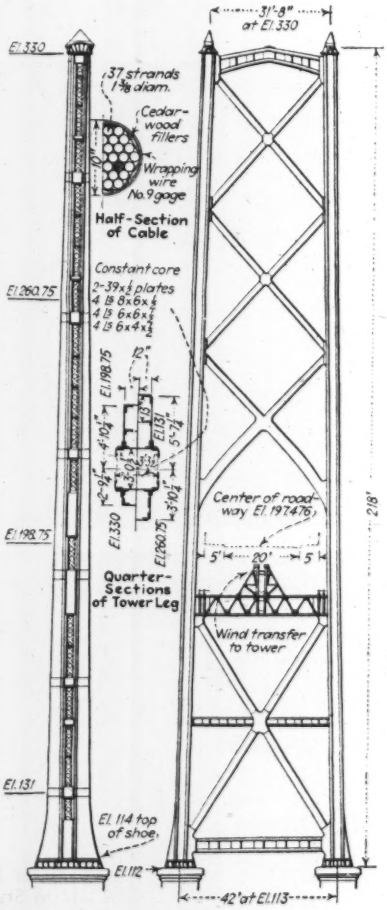


FIG. 5—TOWERS of Ile d'Orleans suspension bridge consist of two legs with a constant core and varying wing sections. They are 218 ft. high, of the fixed-base, flexible type.

wedge-shaped, with the broader base on top, a smooth roadway surface will be provided while the tapered sides will permit ice and snow to pass through readily.

Of special significance is the use of a light-weight concrete and grid floor, about 3 in. thick, the concrete being specified as dry and vibrated, to eliminate any possibility of breaking the adhesion, either from shrinkage or shock. The weight of this floor will be about 50 lb. per sq.ft.

The Ile d'Orleans Bridge is being built by the department of public works of Quebec, of which I. E. Vallée is deputy minister and O. Desjardins is joint chief engineer. The suspension bridge, its flanking spans and the six main piers were designed by Monsarrat & Pratley, consulting engineers, of Montreal, who will also supervise the construction. The substructure contract ($610,000) is held by the Foundation Co. of Canada, Ltd.; and the superstructure contract ($813,000), including the steel of the grid floor, is held by the Dominion Bridge Co., Ltd.

Prevailing Salaries Outlined by Civil Engineers Society

Report based on countrywide figures adjusted for regional and local living conditions presents appropriate salaries for various classes of engineers

A SCHEDULE of prevailing rates of pay for the principal classifications of employed engineers, outlined in a special report to the committee on salaries of the American Society of Civil Engineers, has been approved by the board of direction of the society. The report, by Arthur Richards of the committee on salaries assisted by the headquarters staff of the society, was prepared in response to many requests for an authoritative statement as to prevailing salaries and wages for engineers, both to aid those who are working on wage schedules for the guidance of regional code authorities under a code for the engineers' division of the construction industry (when approved) and also to assist those who are endeavoring to obtain better pay for engineers employed on engineering work under the CWA. The report follows:

Method

The salaries paid 16,046 engineers and engineering assistants in the highway departments of the 48 states in 1930 are taken as the basis of the calculations. From numerous plottings there were determined "key" salaries. Fifty per cent of the engineers received this key salary or more. Six classifications of positions were adopted and the key salary for each was determined. Combining these, a schedule of salaries for the different classifications was set up as an index. The relation of other conditions to conditions surrounding that index schedule permits the calculation of the present prevailing rates of salaries.

Regions

Local conditions of ease or cost of living permit the division of the United States into regions over which these, or other, conditions indicate a variation in the salaries paid. The Bureau of Public Roads of the U. S. Department of Agriculture recognizes nine such regions, and these same regions show sufficient unformity in the salaries of civil engineers in the highway departments to permit them to remain unchanged in this study. These regions are:

New England—Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut.

Middle Atlantic—New York, New Jersey, Pennsylvania.

East North Central—Ohio, Indiana, Illinois, Michigan, Wisconsin.

West North Central—Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska, Kansas.

South Atlantic—Delaware, Maryland, Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida.

East South Central—Kentucky, Tennessee, Alabama, Mississippi.

West South Central—Arkansas, Louisiana, Oklahoma, Texas.

Mountain—Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, Nevada.

Pacific—Washington, Oregon, California.

Classification of positions

Information relating to the duties, responsibilities and salaries of the engineers of the highway departments permits a relatively simple classification of positions which further have been taken as the basis for salaries of civil engineers engaged in other than highway work. These classifications, comparable to the key salaries, are defined as follows:

Engineering Assistant

Duties: To assist on engineering work in field or office; make simple computations, sketches and tracings; perform the usual work of a rodman or chainman.

Requirements: Education equivalent to graduate from a high school with at least two years of experience in office or field assisting on engineering work.

Junior Engineer

Duties: To perform technical work in the office or field requiring a knowledge of engineering practices and methods; make working drawings and designs and prepare estimates; make computations of a technical character; perform minor surveys and construction inspection. This classification includes the position usually called levelman and the lower grades of draftsman and inspector.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus about two years of experience on engineering work; thorough knowledge of care, operation and adjustment of engineering instruments; ability to design simple engineering structures.

Junior Assistant Engineer

Duties: To perform engineering work in office or field requiring the exercise of engineering judgment and skill; to be responsible for the surveys, plans, inspection or work assigned to him by engineers of a higher grade; to take charge of a field party, a drafting squad or inspection of construction. This classification includes the position usually called transitman and the higher grades of draftsman and inspector.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus about three years of experience in engineering work; ability to plan operations in office or field.

Assistant Engineer

Duties: To perform important engineering work in office or field requiring the exercise of independent engineering judgment and skill; to supervise designs, drawings and specifications; to make studies, estimates and reports; and to supervise the work of one or more field parties or a group of inspectors on construction work. This classification includes the position usually designated as designing engineer.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus at least four years of experience in engineering work of which at least one year shall have been in responsible charge; eligible for corporate membership in one of the national engineering societies or for license as a professional engineer; experience in surveying, design, construction or maintenance of engineering projects.

Engineer

Duties: To have charge of a minor engineering organization, or to have immediate administrative and technical charge of a division in a large organization (comprising several squads or parties in office or field); to plan, direct and supervise the design or construction of engineering projects; to supervise the preparation of contracts and specifications and to be entirely responsible for the engineering, construction and busineer operations of his division, bureau, section or department. This classification also includes the position usually designated as assistant chief engineer.

Requirements: A first-class technical education plus about eight years of professional experience in the active practice of surveying, design, construction or maintenance of engineering projects or structures.

Chief Engineer

Duties: To have entire charge of and be responsible for entire organizations comprising more than one engineering division and engaged in more than one class of work; to determine technical

and administrative policies and procedures; to be responsible for all engineering construction and business operations; and to investigate all enginereing projects submitted for consideration and report regarding feasibility, cost and economic necessity.

Requirements: A first-class technical education plus about fourteen years of broad professional experience embracing a thorough knowledge of engineering office methods, field surveys, design, supervision of construction, maintenance operations, materials of construction and general business administration.

Index schedule

The index schedule combines the classifications of positions with the key salary for each classification. Salaries derived from the 1930 survey are shown by the later surveys to have suffered, in common with incomes from other sources, reductions differing locally but in general variable with relation to their size, i.e., the larger reductions being observed in the higher salaries. Fifty per cent of the engineers in each position classification in the schedule received the salary shown, or more. The following is set up as the index schedule showing both 1930 salaries and the appropriate reduced salaries as of March 1, 1934:

	1930	1934
Engineering assistants.	Not less than	$1,500
Junior engineers	$1,824	1,620
Junior assistant engineers	2,280	2,000
Assistant engineers ..	3,000	2,600
Engineers	4,200	3,600
Chief engineers	7,000	5,700

Regional factors

To the indicated key salaries there should be added or subtracted amounts indicated by the regional factors. Further, to these figures, when so adjusted, other factors are applicable, dependent upon other conditions.

The regional factors were found to be as follows:

New England	plus 3 per cent
Middle Atlantic	plus 10 per cent
East North Central..	plus 6 per cent
West North Central.	minus 2 per cent
South Atlantic	minus 6 per cent
East South Central..	minus 9 per cent
West South Central.	plus or minus zero
Mountain	minus 7 per cent
Pacific	plus 8 per cent

Population factor

The surveys indicate that for those employed in cities of 1,000,000 population a further increase in salary is appropriate in the amount of 10 per cent, and that in New York City with a population of approximately 7,000,000 a factor of plus 40 per cent is appropriate. The data available do not indicate the appropriate additions for cities with population between 1,000,000 and 7,000,000, but probably it is somewhat proportional.

Types of employment

The above index schedule (adjusted by regional factor) applies to engineers engaged in work similar to highway work and resident under similar conditions. Data relative to 4,700 engineering positions of different character justify increased salaries for those employed in certain other types of work or organizations. In general those employed under civil-service protection and with the benefit of pension systems, as in city employ, are found to conform to the adjusted index schedule. For those many selected individuals in other than public employment, such as those in quasi-public employment or in industry, the adjusted key salaries are observed to be properly to be increased up to 25 per cent, and this, as in the cases of highway and municipal employment, is determined on the basis that 50 per cent of the engineers regularly employed receive this adjusted key salary or more.

Example

A civil engineer engaged in highway work and defined as being in the classification of "engineer" is indicated by the index schedule for 1934 to have a salary of $3,600. If he works in the Middle Atlantic region, that figure should be increased by 10 per cent, making it $3,960. An engineer employed in the city of Philadelphia (population 2,000,000 approximately) should receive an additional 15 per cent (population factor) making $4,554, and, if in other type of employment, an additional amount equaling under some methods of employment as much as 25 per cent, totaling $5,692, might be the key salary.

Caution

Statistically the outlined system is believed to be justified, using the key salary as the primary basis. Such a system cannot take into account unusual requirements or conditions nor matters of particular circumstance in respect to those engineers upon whom the greater responsibility is placed. There is no certain method of standard appraisal of the worth of intelligence, knowledge, experience, administrative ability, enthusiasm and activity. The above analysis, however, should afford a statistical basis to which an appraised value of these intangibles may be applied.

Contract Adjustments Made Under the NRA

By Howard L. Jones
Clarksdale, Va.

WHEN the National Industrial Recovery Act and its authorized codes and agreements went into effect, there were in process of execution thousands of government contracts. These contractors found themselves confronted with the problem of mounting prices and increased labor costs. The decisions of the accounting officers and the courts blocked what seemed the only avenue of escape—price adjustments. There was no authority in any officer of the government to modify the terms of a contract with the United States. The price to be paid having been definitely fixed in a legal contract, there was no authority to use appropriated public moneys to pay a greater amount. It must be understood that contracting with the United States involves a situation not applicable to contracting between individuals. As has been well stated in a decision of the Supreme Court of the United States:

> We have no officer in this government, from the President down to the most subordinate agent, who does not hold office under the law, with prescribed duties and limited authority. (7 Wall. 676.)

When a contract has been entered into and the United States has acquired certain rights thereunder, government officers are without authority to waive those rights or to modify the agreement in a manner prejudicial to the interests of the government. It is well understood that every contractor faces the possibility that markets may change during the interval between the date of a contract and the date of its completion. If the change is downward, the profits are increased; if it is upward the profits will be less. These are hazards which every contractor must assume. The United States cannot guarantee that a contractor will earn a profit in the performance of a contract, and its officers have no legal authority to pay any amount in addition to the contract price to absorb a loss. It might as reasonably be assumed that any profits under a contract with the United States should be shared with the government. It was argued by some that as the United States was directly responsible for the increase in cost of performance, through an act of Congress, the price to be paid under a contract with the government should be increased accordingly. This situation is not new in the affairs of the nation. A similar situation was considered by the Supreme Court of the United States, in which it was said that:

> It has long been held . . . that the United States when sued as a contractor cannot be held liable for an obstruction to the performance of the particular contract resulting from its public and general acts as a sovereign. (267 U. S. 458.)

Equities recognized

There are times, such as these, when fluctuations are so abnormal as to be disastrous to business. Here was a national emergency that called for drastic measures to restore normalcy. The remedy in some few cases quickened the ailment. Those contractors who had figured how to keep plants going and men employed found themselves in a difficult financial situation.

On the one hand were codes and agreements, formulated and put into effect by the government, under which they must reduce hours and pay minimum wages. On the other hand were the decisions of the "watch dogs" of

the Treasury, under which there was no authority to change the price to be paid under an agreement with the United States. The equities of the situation were recognized by everyone, but a long line of legal precedents stood in the way of payment as an increase in the contract price.

The President expressed his views in the matter in a statement issued Aug. 6, 1933, as follows:

> The United States government as a buyer of goods should be willing itself to take action similar to that recommended to private buyers. Therefore, wherever adjustments can be made under existing law, I shall recommend that they be made. In other cases, where such adjustments cannot be made under authority now possessed by the executive departments, I shall recommend that the next Congress . . . take action giving authority to the executive departments, under such safeguards as Congress may approve, and making any necessary appropriations to provide for recompensing such buyers who have in good faith and wholeheartedly cooperated with the administration of the National Industrial Recovery Act and as a result thereof should equitably be allowed an increase in the price of goods furnished in the interim in accordance with the terms of contracts entered into with the government prior to June 16, 1933.

Three resorts possible

It may be said that there have been used in the past three methods of adjusting equitable claims against the United States. One method is by a general relief act of Congress authorizing the administrative offices to consider the facts in each case and make an equitable adjustment. This plan is followed usually when the number of claims is so great that individual relief bills would be cumbersome. The Dent Act (40 Stat. 1281), authorizing the cancellation and adjustment of World War contracts, is an example of this type of relief measure. Another method that has been taken advantage of for many years is the preparation of private relief acts by the administrative offices for submission to Congress. This method has been used generally in isolated cases, although a great many such bills are presented at each session.

The third method is of more recent origin, having been authorized in the act of April 10, 1928 (45 Stat. 413), as follows:

> That when there is filed in the general accounting office a claim or demand against the United States that may not be lawfully adjusted by the use of an appropriation theretofore made, but which claim or demand in the judgment of the Comptroller-General of the United States contains such elements of legal liability or equity as to be deserving of the consideration of the Congress, he shall submit the same to the Congress by a special report containing the material facts and his recommendation thereon.

This plan, because of the necessity for Congressional action, may not afford prompt relief, but it has been taken advantage of in a great many cases. The matter of whether or not the equities are sufficient to warrant submission to Congress has been left with the Comptroller-General. Many of the cases submitted have not been considered as warranting Congressional consideration.

Some examples of cases submitted with favorable recommendations may be cited:

1. A claim was made for increased cost alleged to have been occasioned by delays because the government did not supply the contractor with information regarding the location of foundations. It was held that although no adjustment could be made under the terms of the contract the facts presented equities in favor of the claimant.

2. In another case the contractor visited the site of the work and was informed that he would not be required to perform certain work, although the specifications called for the performance of that work. His bid did not include the work in question. Favorable action was recommended, as the work had been performed in accordance with the specifications.

The Comptroller-General has authority under the law (Sec. 305 of the Budget and Accounting Act, 42 Stat. 24) to settle all legal claims for which an appropriation is available without submission to Congress.

It is understood that the President took action in an executive order of Aug. 9, 1933, to insure the filling of contracts with the government by formulating a plan of relief for those who, on account of increased costs resulting from the operation of codes or employment agreements, are not otherwise able to complete their contracts. The mere fact that a contractor sustained a loss would not, it has been held, entitle him to relief under this plan.

All inquiries regarding relief measures should be addressed to the administrative office of the federal government that made the contract.

Letters to the Editor

Sewage Treatment at Schenectady

Sir—In your issue of Nov. 2 we notice an editorial concerning pressure to have the operation of the sewage-treatment plant discontinued on the score that it represents an item of expense that can be eliminated without apparent hardship to the community in Schenectady. It is probable that you have been partly misinformed concerning this matter, particularly in your statement "Deliberately to bring about further river pollution would be a retreat from civilization." This organization has done more than any other organization of which we have knowledge to bring about a movement for the general sanitation of all the cities along the Mohawk.

Schenectady is now treating 60 per cent of its sewage, discharging the other 40 per cent into the river raw. The river both above and below the city is unfit for swimming or for water consumption.

Only one small community [italics ours—Editor] now takes water from the river below this city. Schenectady has spent about $1,000,000 during the last seventeen years in treating a portion of the river's source of pollution. Other cities have spent nothing until one year ago, when one of the smaller cities also began treating its sewage. Schenectady's expenditure has not made the river usable for any of the normal purposes to which sanitary effort usually contributes.

The position of our organization is that if this million dollars had been applied to benefit the community health in other ways, it would probably have prevented more sickness, contributed to more convenience and happiness, saved more lives, and generally given greater benefits than has come from this one-community and relatively futile effort to give a heavily polluted stream some sanitary utility.

Our organization has been at the forefront in advocacy of advanced sanitary and health measures, but its enthusiasm for modern advancements in these lines has not blinded it to misexpenditures through well-meaning but misguided public policies.

Schenectady, N. Y., Nov. 16, 1933. ABBETT PULLIAM,
Managing Director,
Schenectady Bureau of Municipal Research, Inc.

MR. PULLIAM'S argument here is the same as that we objected to in the editorial in question: that sewage treatment in Schenectady is unnecessary, because the Mohawk River is already badly polluted by numerous other towns that make no effort at sewage treatment. Quite evidently the discontinuance of treatment at Schenectady would still further pollute a river already "unfit for swimming or for water consumption." If this were a condition that would present no hazard to public health, the course advocated by the municipal research bureau would have more merit. But the river *is* used, for both swimming and water supply, even if by "only one small community." In all reason, then, the additional health hazard resulting from increased river pollution should be balanced against the additional safeguards to health that could be undertaken with the money saved through discontinuance of plant operation. How much this saving might be is not stated in Mr. Pulliam's letter; in view of the continuing character of many elements of the total cost of sewage treatment, it is not likely that discontinuance of operation would result in a saving large enough to inaugurate any important public-health measures not already in effect. It is to be noted that in regard to the "other ways" of spending the money Mr. Pulliam's letter is quite vague. If no new protective measures are in fact contemplated as alternates to sewage treatment, the course recommended by the bureau appears to be merely a sacrifice upon the altar of economy. EDITOR.

ENGINEERING NEWS-RECORD

Volume 112, No. 11 Copyright 1934 by McGraw-Hill Publishing Company, Inc.

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Editorial and Publishing Offices at 330 West 42d Street, New York

March 15, 1934

Sale of Municipal Bonds

FOR THE FIRST time in its history the federal government has sold municipal bonds to the private investment market. As reported in our news pages this week, the Public Works Administration on March 6 sold at a substantial premium four blocks of municipal bonds acquired as security for PWA loans. In a sense this action is consistent with the interpretation of economic conditions made several years ago by the proponents of federal stimulation of local public construction. Improvement programs of local governments, it was held, were blocked by the timidity of private capital, and it was proposed that the federal government, as a measure of economic recovery, lend its credit to finance local work until private funds became less retiring. The response to the PWA's offering, and indeed the course of the bond market during the past month or so, indicates that private investors are again becoming receptive to municipal bonds. But now that municipalities are loath to borrow money, however eager investors might be to lend it, federal participation in the form of a partial subsidy is essential if sufficient activity is to be created to aid business revival. A quickening market for municipal bonds under these conditions does not by any means imply that the ministrations of the PWA are no longer needed.

Disappointing?

ANNOUNCEMENT by the President that no portion of future funds now budgeted for public works will be specified for highway improvement calls for a sounder reason than mere assertion that the present $400,000,000 has been disappointing in the amount of employment produced. Despite the six weeks' starting handicap imposed by the public-works administrator while he fussed with rules and regulations and state programs, the trained organizations of the Bureau of Public Roads and of state highway departments have contracted and completed more work and given employment to more workers than has been even attempted by any other of the PWA agencies. This statement is demonstrable by figures which no pronouncement even from the highest places can alter. Instead of expressing disappointment with its highway agencies the Public Works Administration should be thankful that it has had this saving exception to its general failure to get work started and men re-employed.

The Housing Pot Boils

DEVELOPMENTS in the drive to secure low-cost housing and to eliminate city slums continue to follow one another at a rapid rate. Tangible results seem exasperatingly elusive, but if change can be called progress the future is not without hope. The obstacles which the Comptroller-General reared in the path of the Public Works Emergency Housing Corporation were removed last week by a ruling from the Attorney-General. The same corporation had its assets swelled by some $23,000,000 when allotments of that amount previously made to limited-dividend companies were rescinded and transferred to its account. This latter is progress in the sense that it places the money where it is likely to be utilized more quickly. In New York, condemned tenement houses are being vacated, and plans are under way to force repairs and modernization or demolition. Demolition also is being fostered widely by the CWA, which has made it a preferred type of project; it is without doubt effective slum clearance and fits more ideally the CWA requirements of maximum labor-material ratio and socially useful character than anything so far tried. Low-cost housing and slum clearance may yet be made to do their part in the recovery movement.

Three-Dimensional Blueprints

IN THE PRESENT widespread preoccupation with model studies for determining the behavior of structures under loads we are apt to forget one of the original purposes of models, which was to explain to the craftsman how the designer's brainchild was to be assembled. Models were, in effect, three-dimensional blueprints. This ancient use for a model was revived by the engineers who designed the new suspension bridge that is being built over the St. Lawrence River at Quebec. Their problem was to devise a sequence of construction operations for the anchorages which, first, would quickly place the early stages of the work out of the danger of floods and, second, would permit efficient coordination of the work of the contractor who pours the concrete and the contractor who erects the cables. A scale model in wood was made in which each pour of concrete was represented by a separate block, and each block was numbered in the order of placing. Both contractors have used the model in planning their operations. Such assembly models offer an excellent medium for effective cooperation between designer and builder.

New Knowledge Available

CIVIL ENGINEERS and aeronautical engineers are strangers to each other. This is unfortunate because both are vitally concerned with a common problem—namely, the idiosyncrasies of wind as they relate to the stability of structures. The civil engineer is the greater loser from this lack of acquaintanceship because it is the aeronautical engineer who has utilized modern advantages in the way of scientific apparatus and money to delve deeper into the subject of wind action. While the civil engineer has been satisfied with an accepted approximate relation between wind velocity and pressure that has undergone the refinement of 150 years since Isaac Newton, the aeronautical engineer perforce has had to learn more about the characteristics of air flow and the possible variations in pressure as affected by such factors as size and shape of structure. However, the new knowledge which he evolved was disguised in a language unfamiliar to the civil engineer and until now uninterpreted for him. In this issue W. W. Pagon, who has had a rare combination of experience in both structures and aerodynamics, undertakes this interpretation and in future issues will describe the application of some of the new scientific knowledge to civil engineering problems. Such aerodynamic terms as "stagnation

A Program of Action

EFFECTIVE PROSECUTION of the recovery campaign under the critical circumstances in which the nation is now involved requires a program of determined and consistent action. As we pointed out a week ago, such action centers in vigorous continuation of the public-works movement through a new program starting from the accumulated four-billion-dollar pool of PWA projects now left unprovided for. But the effectiveness of this plan depends wholly on the speed with which the program is made to produce employment and business stimulation.

The fatal slowness that brought the public-works program of the Recovery Act to failure is known to everyone. In almost startling contrast, the Civil Works Administration succeeded in getting several million men to work within a month, and thereby saved a threatening winter situation. To make the 1934 public-works program effective for recovery, a combination of CWA speed with the soundness of PWA projects is the first essential. For it is quite evident that progress toward recovery is measured less by the size of the program than by the rate at which its dollars are transformed into useful work. Money spent a year or two hence cannot re-employ the idle worker today or start up the idle factory.

Fundamental to the impotence of the Public Works Administration, as previously of the Reconstruction Finance Corporation, is its banker attitude toward its mission of initiating construction. If there is to be any progress, this attitude must be eliminated and replaced by the aim to bring about recovery and re-employment. It has led to countless delays and failures of projects, and by no possibility can the civil-works slack be taken up if it continues to prevail. The "reasonable security" which the law stipulates surely was never intended to mean gilt-edged bonds. Congress must make its intent unmistakable if emergency financing of public works is to become a reality.

Other reforms in great number are needed in the PWA procedure, though most of them are derived from this one. Its rigid labor and wage limitations must be relaxed. Its centralization of all authority at Washington, which has made the state PWA engineers and boards little more than errand boys, must give way to equally thorough decentralization—the secret of CWA success. Obstructive detail practices in great number that grew up out of the desire to conduct an investment business instead of a recovery and re-employment campaign are in urgent need of correction.

Here lies the immediate duty of the Congressional leaders and the committees which bear the responsibility of shaping the 1934 public-works campaign. It is their task to establish an administration that can carry out a program of action. Fortunately, engineers and city officials in every corner of the country have had occasion to learn by their own bitter experience the sources of fatal delay in the present PWA system. Their testimony is adequate to guide Congress to success in its task of inaugurating a program of effective recovery action based on public works construction.

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point," "drag coefficient," "boundary layer" and "eddy frequency" will be seen to be useful additions to civil engineering language and learning. They will permit a better understanding of the nature of wind forces, of the non-uniformity of pressure, of the existence of large negative-pressure or suction areas, and of the twisting moment on structures in high winds. Civil engineering must not too long remain ignorant of the results of modern aerodynamic research.

Pay for Engineers

FORMULATION of a wage and salary schedule for engineers by the American Society of Civil Engineers is a courageous and much-needed action. Engineers everywhere are faced with the question of what are "prevailing" rates of pay for employed members of the profession. The question came up early last summer in connection with the PWA work; later it was an open issue in the code discussions; still later it arose under the CWA; and it is with us today. At present the local sections of the Am.Soc.C.E. and other regional groups are working upon it in anticipation of the establishment of regional schedules by local code authorities under the proposed engineers' code.

There is no such thing as a "prevailing wage for engineers," as that term is used in relation to skilled mechanics. If such a rate existed, there would have been no need for the long study undertaken by the society's committee on salaries, nor for the summary of findings of fact that was prepared as a background for the schedule drawn up. Engineers in the same region are paid one rate in civil service, other rates by public utilities, and a wide variety of rates by consulting engineers, contractors and others. Out of this confusion of practice, regional code authorities soon will have to select rates that can justly be set up as a minima for the principal definable grades of engineering service, as a step toward the agreements between truly representative groups of employers and employees which the code system contemplates.

It is for this reason that promulgation of a prevailing wage schedule by the American Society of Civil Engineers is a useful, timely service. Coming in advance of the start of minimum-wage studies by regional code authorities, it promises to reduce the difficulties with which these authorities will have to contend. It is to be expected that the schedule rates will prove to be inequitable in some instances, but they have this in their favor: they represent the actual rates paid to at least 50 per cent of one of the largest groups of employed engineers scattered throughout the entire country—the highway engineers. If any salary rates have a strong claim to be considered as prevailing, it is these rates.

Many will question the wisdom of the society's definite and positive action in promulgating these rates with the approval of the board of direction, just as many have questioned its aggressive action in the matter of the engineers' code and its earlier action in the matter of stimulating public construction as an aid to recovery. But the times call for aggressive action. In the wage and salary matter nothing is better calculated to bring out protest from those who may fear injury by the proposed rates, and where protest is found justified an adjustment can quickly be reached.

CURRENT NEWS

Municipal Bonds Sold by PWA At a Premium

MUNICIPAL bonds with a total par value of $737,000 were sold by PWA this week at premiums to the highest bidders for $757,050.19. All the issues offered were 4 per cent, with one exception, and the PWA received 31 bids, 29 of which were at premiums. Two bids for $100,000 Bel Air, Md., 4 per cent sewer bonds were rejected.

The successful bidders and the price bid on the issues which comprise the first offering by the PWA of bonds acquired in financing local public works projects were as follows:

$125,000 Cedar Rapids, Ia., 4 per cent sewer bonds due Jan. 1, 1937-41, to White Phillips Co., Davenport, Ia., for $127,839.

$250,000 Erie County, N. Y., 4 per cent road and bridge bonds due Oct. 15, 1935-36, to Salomon Brothers & Hutzler, New York, for $250,010.

$200,000 Lynchburg, Va., 4 per cent water supply bonds due Nov. 1, 1936-50, to Brown Goodwyn & Co., Washington, D. C., for $203,530.

$162,000 Richmond County, Ga., 4½ per cent school bonds due Jan. 1, 1935-60, to Chase National Bank of New York for $175,671.19.

Bids on $100,000 Bel Air, Md., 4 per cent sewer bonds due Oct. 15, 1934-50, were made by W. W. Lanahan & Co., Baltimore, for $97,031.25 and by G. M.-P. Murphy & Co. and Equitable Trust Co. of Baltimore for $91,345.86.

PWA Administrator Ickes characterized the sale as establishing the broader conception of the public works theory of aiding employment to speed recovery. The bids show, he said, that in the instances involved private sources are now awaking and willing to assume their proper function while the procedure followed by the government created employment on the projects financed and returned a profit to the government.

Minnesota Governor Appoints State Planning Board

A state planning board of seventeen members, many heads of departments, has been appointed for Minnesota by Governor Olsen to outline social, economic and educational advancement. Engineers on the board are the following: E. V. Willard, commissioner of conservation; N. W. Elsberg, highway commissioner; H. A. Whitaker, engineer, State Board of Health; F. M. Mann, Minnesota City Planning Commission; George H. Herrold, St. Paul City Planning Board; A. B. Horwitz, Duluth City Planning Commission, and four professors of the University of Minnesota, not engineers, O. B. Jesness, R. G. Blakey, R. E. Scammon and M. B. Lambie (coordinator Minnesota CWA).

Army Engineers Estimate Damages From Canalization

The chief of engineers of the United States Army sent to the Rivers and Harbors Committee of the House of Representatives a report on the estimated damage that would be done lands by the $124,000,000 P.W.A. project for improvement of the Mississippi River between the Missouri River and Minneapolis.

It was estimated that the equivalent capitalized value of the damage at 4 per cent would be about $1,744,000 and at 6 per cent, $1,183,000. The chief of engineers pointed out, however, that these damages would be small with respect to the project as a whole, but serious to the districts affected, under existing economic conditions. He recommended that Congress authorize payment of just compensation for such damage when incurred.

The report further pointed out that the district engineer had estimated that the expected damages may be remedied by construction of additional pumping plants at a cost of $40,000 and by increased operation of existing and new drainage units at an estimated annual cost of $71,000.

Eastman Urges Federal Regulation Of All Transport

PRESENTING the second of a series of reports to Congress and the President on what is wrong with the transportation industry, Transportation Coordinator Eastman on March 10 recommended that motor trucks and buses and water transportation be brought as soon as possible under federal control.

In his first report several weeks ago Mr. Eastman declared government ownership and operation of railroads was the only solution to the problem, but held that the time had not yet come for such a move. In this second report he strongly urged that there is need for federal legislation to regulate other transportation agencies and the railroads and to promote proper coordination of all means of transportation, and that there is need for amendments to federal statutes to improve the present system of regulating the railroads.

(Continued on p. 368)

Seven Housing Allotments Rescinded; Transferred to Federal Housing Corp.

THE SUM of $23,670,500, previously allotted to seven low-cost housing and slum clearance projects, has been transferred to the Public Works Emergency Housing Corp., bringing the amount that corporation now has available for housing and slum clearance projects to $123,670,500.

In order to make the transfer it was necessary to rescind seven housing allotments previously made.

The allotments rescinded are as follows:

The Spence Estate Housing Corp., Brooklyn, N. Y., $2,025,000.

Hutchinson Suburban Housing Association, Hutchinson, Kan., $40,000.

Cleveland Homes, Inc., $12,000,000.

Indianapolis Community Plan Committee, Indianapolis, Ind., $4,460,000.

Techwood, Inc., Atlanta, Ga., $2,600,000.

University Housing Corp., Atlanta, Ga., $1,212,500.

Harms Park Housing Corp., Chicago, Ill., $1,333,000.

In every case, except that of Hutchinson, Kansas, the allotments were rescinded because the local interests sponsoring the project failed to meet the PWA requirements as to equity after the allotment was made. In the case of the Hutchinson Suburban Housing Association the applicant failed to execute the contract.

The PWA has announced that the rescinding of these allotments does not mean that all these projects have been abandoned. Some may be constructed either by the Housing Corporation or by the Administrator.

Allotments for 117 non-federal projects aggregating $4,418,837 were announced during the week by Public Works Administrator Ickes. These new allotments bring the total of PWA allotments for qualified non-federal projects to $882,247,638.

Funds for this new list were obtained by rescinding earlier allotments which applicants were unable to utilize. Sixty-seven of the allotments were for loans and grants while the remaining 50 were for grants only.

The PWA announced also changes in several previously made allotments to non-federal projects, the largest of which was an increase to $2,500,000 of a loan and grant of $1,500,000 previously allotted to Salem, Ore., for waterworks construction. At the time the original allotment was made some months ago the city expected to acquire the present privately owned water system and improve it to better serve the residents of the city. The first allotment was to cover the cost of remodeling and improving the system after it had been acquired. However, the city has been unable to purchase the old system on satisfactory terms and has decided to build a complete new system, which the increased allotment will enable it to do.

The changes, including that relating to Salem, result in a net increase in allotments of $806,900.

Several allotments previously announced have been rescinded during the week because the applicant was unable to proceed with the construction provided for. The largest of these rescinded allotments was a $3,000,000 loan and grant to Danville, Va., for the construction of a hydro-electric plant. Danville voters at a recent election refused to authorize the project.

Organization Set Up To Coordinate Regional Planning

ANOTHER STEP in national planning was taken on March 11 when Public Works Administrator Ickes announced the appointment of District Chairmen to assist in the coordination of interstate and regional planning proposals and studies into a carefully considered long-range program. Upon the recommendation of the National Planning Board the Administrator has divided the country into twelve planning regions, grouping states from the standpoints of primary drainage, land use and transportation interests. Appointment of District Chairmen for eight of the regions has been announced. The others will be named later.

Formal organization of regional planning work already has been established in New England and in the Pacific Northwest. The following appointments were announced:

George L. Radcliffe, Baltimore, Md., District No. 3, composed of Maryland, Virginia and North Carolina.

Henry T. McIntosh, Atlanta, Ga., District No. 4, composed of South Carolina, Georgia, Alabama and Florida.

Alfred Bettman, Cincinnati, Ohio, District No. 5, composed of Ohio, Indiana, Kentucky, West Virginia and Tennessee.

Charles M. Moderwell, Chicago, Ill., District No. 6, composed of Michigan, Illinois, Wisconsin, Minnesota and Iowa.

Vincent M. Miles, Fort Smith, Ark., District No. 7, composed of Arkansas, Louisiana, Mississippi and Oklahoma.

J. C. Nichols, Kansas City, Mo., District No. 9, composed of Missouri, Kansas, Nebraska, Wyoming and Colorado.

Marshall N. Dana, Portland, Ore., District No. 11, composed of Washington, Oregon and Idaho.

Duncan McDuffie, San Francisco, Calif., District No. 12, composed of California, Arizona, Nevada and Utah.

The New England States will comprise District No. 1; New York, New Jersey, Pennsylvania and Delaware, District No. 2; Montana, North Dakota and South Dakota, District No. 8 and Texas and New Mexico, District No. 10.

It will be the duty of the District Chairmen to stimulate interest in city, state and regional planning and to assist the National Planning Board to coordinate state planning. They will be the contact between the National Planning Board and state planning units, official or voluntary.

Messrs. Radcliffe, McIntosh, Moderwell, Miles and Dana were regional advisers in PWA up until the regional advisers offices were discontinued on Feb. 28.

San Gabriel No. 2 Dam Work Stopped to Allow Settlement

Delay of four to six months in the completion of the San Gabriel Canyon No. 2 rock-fill dam, under construction by Los Angeles county on the west fork of the San Gabriel River, has been ordered by the county board of supervisors to allow time for the structure to settle before final portions of concrete slab are placed on the upstream side.

Earthquake Causes Damage In Utah and Idaho

On March 12 severe earthquakes shook northern Utah and southern Idaho and lighter temblors were reported in sections of Nevada, Wyoming and California. A number of buildings were damaged in Salt Lake City and in Logan, Utah, where the walls of a building at the State Agricultural College were cracked.

In Salt Lake City three shocks caused the closing of all city schools and the evacuation of many office buildings as a precautionary measure. No injuries were reported and damage to buildings was minor. The first and sharpest shock came at 8:05 a.m. Plaster in many buildings was cracked and residents from all parts of the city reported rattling dishes, swaying trees, and in some instances cracked tile. The second tremor came at 8:32 a.m. and the third at 11:21 a.m.

According to Frederick J. Pack, University of Utah geologist, the center of the disturbance was in the north end of Great Salt Lake, near the town of Kelton.

Prof. Perry Byerly, of the University of California, described the earthquake as being as severe as the shocks in Santa Barbara, Calif., in 1925. The records indicated that the shock was considerably heavier than the destructive Long Beach, Calif. earthquake of March 10 last year.

Washington Notes

By Paul Wooton
Washington Correspondent

The appropriation bill covering emergency expenditures for this fiscal year and next for public works and other activities incident to the recovery program is still under preparation in the budget bureau.

In connection with the statement attributed to the President that no further appropriation should be made for road construction, Secretary Ickes by letter on March 12 requested Thos. H. MacDonald, chief, Bureau of Public Roads, to discuss with him "what if anything further ought to be done" in the way of a road program. Mr. Ickes said he had no knowledge of any opinion the President might have on the subject.

Secretary Ickes served an ultimatum on Governor Bryan of Nebraska, Dec. 13, regarding the latter's obstructive tactics with reference to water rights agreements involved in the Sutherland and Loup projects, stating that unless progress were soon made the PWA allotments would be rescinded.

New PWA allotments of the week include $386,000 for purchasing a 386-acre tract for the National Arboretum in Washington and $6,000 to cover professional services in planning the grouping of public

BRITISH VERTICAL LIFT BRIDGE RIVALS LARGEST OF ITS TYPE

The new bridge over the Tees, at Newport, England, opened on Feb. 28, is unique in that it is the first vertical lift bridge to be constructed in the United Kingdom, and is among the three largest of its type in the world, its 2,700-ton lift span being only 30 tons lighter than the duplicate bridges over the Hudson River at Albany and Troy, N. Y., completed last year; the span of these bridges is, however, 341 ft. compared with 274½ ft. for the British bridge. A 38-ft. roadway and two 9-ft. sidewalks are provided. When open, the lift span affords a shipping clearance of 120 ft.

The trusses, 40 ft. 3 in. deep at midspan, are set 43½ ft. on centers. The bridge roadway was designed for a load of 100 tons carried on four wheels plus 50 per cent impact considered together with a uniform load of 150 lb. per sq.ft.

Each tower is carried on four cylinders sunk from 75 to 90 ft. below high water. The front cylinders are 18 ft. in diameter except the bottom 24 ft., which is 27 ft. in diameter. The bottom lengths are steel caissons sunk under compressed air while the upper lengths consist of cast-iron rings made up of segments bolted together and filled with concrete as sinking proceeded. The rear pairs of cylinders are of similar but lighter construction. Each pair of cylinders is braced apart with steel framing at high water level. The distance between the front and rear pairs of cylinders is 61½ ft.

The lift span is operated by two 32-hp. electric motors. A 450-hp. gasoline engine is installed for standby use.

The bridge was built by Dorman, Long & Co., Ltd., as a joint project of the Ministry of Transport, the Durham County Council and the Middlesborough Town Council.

buildings in Cincinnati. The National Planning Board will assign a consultant to assist the Cincinnati city planning commission in the work.

The articles of incorporation of the Public Works Emergency Housing Corp. probably will be amended in conformity with the Attorney General's recent opinion. At present the corporation is proceeding to acquire options through local representatives in various cities. Secretary Ickes said the names of the cities would not be made public, as a defense against land speculators. The Detroit housing project will not be considered, said Mr. Ickes, until the local commission can assure him that land can be purchased at a fair price.

David E. Lilienthal of the TVA stated on Dec. 13 that construction of a duplicate power distribution system at Knoxville apparently would be necessary since negotiations with the Electric Bond & Share Co. have not progressed satisfactorily. Following a conference with Mr. Lilienthal Secretary Ickes said he had discussed PWA cooperation with TVA on municipal projects in Knoxville and several Alabama towns. An allotment of $2,500,000 made by the PWA for Knoxville is still available and no new allotments are said to be contemplated.

Report on Beaver and Mahoning Rivers

The largest project recommended by Major-General E. M. Markham, Chief of Engineers, to the Rivers and Harbors Committee, is the expenditure of $37,000,000 on the Beaver and Mahoning Rivers in Pennsylvania and Ohio. The program would provide for a 12-ft. channel from the mouth of the Beaver River to Struthers, Ohio, with a width of 250 ft. in the Beaver and 200 ft. in the Mahoning River, increased to 300 ft. at bends; construction of dams with twin locks, 56 by 300 ft. and dredging in accordance with plans submitted by the district engineer.

Approval was conditioned on local interests furnishing, free of cost, to the United States all necessary rights of way and flowage rights, except those necessary for reservoirs; suitable areas for disposal of dredged material; guarantees to the United States against all damage claims; construction, maintenance and operation of all bridges and highways needed for the improvement. Local interests would also be required to operate the Milton reservoir under supervision of the Chief of Engineers for the purpose of increasing low-water flow; and provide at their expense terminals and dredging of harbor areas. The annual maintenance and operation cost of the system was set at $630,000.

Engineering Firm Celebrates Fortieth Anniversary

On March 12 the firm of Ford, Bacon & Davis celebrated the 40th Anniversary of its founding. When it was formed in 1894, by two young men just out of the engineering school of the University of Pennsylvania, the firm entered into industrial electrification. Later its work expanded into nearly every field of planning and construction, as well as valuation, management and financing. Its operations dealt with street railways, power plants, bridges, tunnels, terminals, reservoirs, and a wide range of other enterprises. The firm has directed the investment of half a billion dollars in new undertakings.

Contract Registration Arranged for by Code Authority

THE MOST IMPORTANT action taken at the meeting of the Divisional Code Authority for General Contractors on March 10 was with reference to the registration of construction work and services authorized both under Chapter I and Chapter II—General Contractors Division. Beginning March 19, the effective date of Chapter II, registration will be required of all construction work exceeding $2,000 in value undertaken by general contractors. Registration fees will be charged at the rate permitted in Chapter II of one-tenth of one per cent of the value of the work. The code specifies that from the funds so collected shall be defrayed the expense of administering Chapters I and II among general contractors, including the subdivisional administrative committees for building contractors, heavy construction and railroad contractors, highway contractors and others that may be hereafter established for other subdivisions.

The Construction Code Authority of the basic code having previously decided that registration of construction work and services permitted under Chapter I would be carried out by the divisional code authorities, the registration described will fulfill the requirements of both Chapters I and II so far as general contractors are concerned. The request for registration will be passed along to the Construction Code Authority which will issue a registration certificate for use on the job and a receipt to be returned to the applicant through the division. The pertinent information concerning the project furnished on the application form will be summarized and analyzed to supply statistics required by NRA as to operations in the construction industry. Application forms are being printed and will be distributed through general contractor channels.

The following officers were elected by the divisional code authority:

Chairman—A. E. Horst, of Philadelphia.
Vice-Chairman—A. C. Tozzer, of New York.
Treasurer—E. M. Rust, of Washington.
Secretary—E. J. Harding, of Washington.

The Chairman was President of the Associated General Contractors in 1930, and a member of the Code Committee for General Contractors, and of the Code Committee of the Construction League of the United States.

The Vice-Chairman has just retired as President of the Associated General Contractors, and was Chairman of the Code Committee for General Contractors.

The Treasurer has been actively identified with the group of engineering constructors which has been considering code provisions for the group of firms which both design and construct.

The Secretary has been identified with the Associated General Contractors since it was organized, and has been its Managing Director since 1930.

The complete membership of the Divisional Code Authority is as follows:

*W. F. Austin, President, W. E. Wood Company, Detroit, Michigan (Builders).

**James B. Bray, Member of Firm, W. W. Boxley and Company, Roanoke, Virginia (Heavy Construction and Railroad).

*E. T. Foley, Chairman of Board, Foley Brothers, Inc., Pasadena, California (Heavy Construction and Railroad).

*E. P. Forrestel, President, Cold Spring Construction Company, Akron, N. Y. (Highway).

*Nick F. Helmers, Vice-President, Siems-Helmers, Inc., St. Paul, Minnesota (Heavy Construction and Railroad).

*A. E. Horst, Secretary-Treasurer, Henry W. Horst Co., Philadelphia, Penna. (Heavy Construction and Railroad).

*W. A. Klinger, President, W. A. Klinger, Inc., Sioux City, Iowa (Builders).

**William C. Miller, Partner, W. C. and A. N. Miller, Washington, D. C. (Residential).

*R. E. O'Connor, President, J. C. O'Connor and Sons, Inc., Ft. Wayne, Indiana (Highway).

*E. P. Palmer, Partner, Senior and Palmer, New York, N. Y. (Heavy Construction and Railroad).

*L. E. Ray, Secretary-Treasurer, Diamond Engineering Company, Grand Island, Nebr. (Highway).

**E. Marshall Rust, Vice-President, Rust Engineering Company, Pittsburgh, Pa. (Engineering Constructors).

*F. L. Shackelford, Vice-President & Treasurer, Potter & Shackelford, Inc. Greenville, S. C. (Builders).

**Chas. H. Simpson, President, Municipal Paving and Construction Company, Nashville, Tenn. (Highway).

*A. C. Tozzer, Executive Vice-President, Turner Construction Co., New York, N. Y. (Builders).

**Robert A. Whidden, President. The Whidden Company, Boston, Mass. (Builders).

*H. B. Zachry, President, H. B. Zachry Company, Laredo, Texas (Highway).

The authority is establishing headquarters in the Munsey Building, Washington, D. C. where communications should be addressed.

Progress is reported on drafting the subchapters for the three subdivisions mentioned above, which, upon approval by the Divisional Code Authority, will go to the Construction Code Authority and thence to NRA. The Divisional Code Authority for General Contractors will meet again in Washington on March 24.

(Names marked * are members of the A.G.C.; those marked ** are not members of that organization.)

Diversion of Water Income To Sewage Treatment Upheld

Reversing a Common Pleas Court decision, the Court of Appeals has ruled that the city of Cleveland may continue to divert waterworks income for the maintenance and operation of its sewage treatment plant. The ruling set aside a permanent injunction granted last November to prevent continuance of the city's ten-year-old practice.

The court was divided in its decision, with the majority holding that an ordinance passed in 1924 declared sewage disposal to be a part of the water purification process and therefore an expense to be met, in part at least, by waterworks income.

President Approves Code For Decorating Division

The code for the Painting, Paperhanging and Decoration Division of the Construction Industry—designated Chapter III—was approved by President Roosevelt on March 12 and becomes effective on March 22. The Cement-Gun Contractors Chapter is nearing approval. The Electrical Contracting Chapter, which provides a seventy-five cent minimum rate for electrical workers, is held up pending settlement of a protest by electrical manufacturers against the inclusion of the installation of electrical equipment in the definition of the chapter, although the installation of manufactured parts and products is an integral part of the definition of the construction industry in Chapter I.

Public hearings probably will be held during the week beginning March 19 on the chapter for Tile Contracting Division, Masonry Contracting Division, Heating, Piping and Air Conditioning Contractors Division, and Contracting Plasterers Division, with especial reference to the attachment thereto of agreements between the employers national associations and union labor fixing minimum rates for skilled labor. It is understood that the chapter for the Elevator Manufacturing Industry will go through for approval by mutual consent without the attachment of the labor agreement between this group and union labor.

Additional Crews Put to Work On Boulder Dam Power Line

Early in March a number of additional construction crews were added to the forces erecting steel towers for the transmission line which is to carry power from Boulder Dam to Los Angeles. Of the 2,800 towers required, 300 had been erected on March 1 and the work was progressing at the rate of 12 towers per day. The line is to carry 275,000 volts and is to be completed and ready for service by September, 1935, at which time it is expected that the generating plant on the Colorado River will start to generate power.

Referendum on Sewage Project Held Unconstitutional in Ohio

According to the ruling of H. G. Southard, director of the Ohio State Department of Health, the efforts being made to hold up the construction of the $3,400,000 sewage disposal plant in Columbus by calling for a referendum before contract could be let will be unavailing. Dr. Southard ruled that the orders of the Health Department are not subject to referendum under the Ohio constitution. Referendum petitions were being circulated but those behind the move could not be ascertained. Plans at present call for the opening of bids on the first of the contracts on March 29.

Slum Clearance by Piecemeal Demolition of Old Tenements

SPURRED by the CWA announcement in January that demolition of useless structures and salvage of materials was a proper type of project on which to use federal funds, a number of cities have instituted campaigns looking toward piecemeal slum clearance. In Chicago the Metropolitan Housing Council has initiated a program based on owner requests and on condemnations by the city building commissioner. Although some 4,000 structures in the city are at or near the condemnation stage, the building commissioner has actually only condemned 200 because the city has lacked funds to demolish them as required. A demolition department has been set up in the CWA under Col. Dan I. Sultan, Cook County administrator, to carry out this project.

In Detroit inspections by CWA workers have uncovered large numbers of "unfit for use" structures. In one 10-block East Side area not a single building out of a total of 341 is in good condition: 142 of these structures are beyond repair. The surveys of this site and also of several others in which conditions are nearly as bad are being used as basic information in the selection of a site for a possible slum clearance

DETAILS OF PWA LOANS AND GRANTS FOR NON-FEDERAL WORK

Locality	Loan and Grant or Grant	Type	Amount
Lansing, Ill.	L. & G.	Waterworks impr.	$60,000
Springfield, Ohio	G.	Court building	1,500
Newton, Iowa	G.	Humus tank	3,000
Cavalier, N. D.	L. & G.	Sewer system ext.	3,500
Holly Hill, S. C.	L. & G.	Waterworks system	34,000
Danville, Va.	L. & G.	Electric service	18,000
Cavalier, N. D.	L. & G.	Sewer system	6,000
Cavalier, N. D.	L. & G.	Sewer system ext.	2,400
Cavalier, N. D.	L. & G.	Sewer system ext.	5,000
Zanesville, Ohio	L. & G.	Sewer system	83,000
Danville, Va.	L. & G.	Water system ext.	85,000
Hutchinson, Kan.	L. & G.	Sewer system	104,000
Newport News, Va.	L. & G.	Incinerator	65,000
Gretna, Va.	L. & G.	Waterworks system	43,000
Charlottesville, Va.	L. & G.	Repair to dam	85,800
Willow Springs, Mo.	L. & G.	Water system	32,000
Westchester County, N. Y.	L. & G.	Trunk sewer	28,100
Carroll County, Ga.	L. & G.	Jail building	73,000
White Plains, N. Y.	L. & G.	Water system	51,000
Dillwyn, Va.	L. & G.	Waterworks system	28,000
Iola, Kan.	G.	Swimming pool, bath house and water treatment plant	8,200
East Rochester, N. Y.	L. & G.	Waterworks impr.	80,000
Vernon, N. Y.	L. & G.	Waterworks system	50,000
Hartford City, Ind.	L. & G.	Streets	91,000
Pleasantville, N. Y.	L. & G.	Water main	11,000
Algonac, Mich.	L. & G.	Waterworks impr.	56,000
Batavia, Ill.	L. & G.	Bridge	21,000
Elba, N. Y.	L. & G.	Waterworks impr.	48,000
Dayton, Ohio	L. & G.	Bridge repairs	81,900
Big Stone Gap, Va.	L. & G.	Concrete dam	63,000
Mt. Lebanon, Pa.	L. & G.	Sewers, sidewalks, etc.	40,000
Mt. Lebanon, Pa.	L. & G.	Streets	38,000
Mt. Lebanon, Pa.	L. & G.	Park improvements	41,000
St. Marys, Pa.	L. & G.	Streets	82,181
Stringtown, Okla.	L. & G.	Waterworks impr.	34,000
Painted Post, N. Y.	L. & G.	Street impr.	45,000
Ellwood City, Pa.	L. & G.	School addition	100,000
Petersburg, Va.	L. & G.	Water and sewer ext.	89,000
Old Lyme, Conn.	L. & G.	School	100,000
Cheyenne Wells, Colo.	L. & G.	Waterworks impr.	11,000
Bluefield, Va.	L. & G.	Sewer ext.	49,000
Fairport, Ohio	L. & G.	Waterworks impr.	95,000
Kilmarnock, Va.	L. & G.	Waterworks system	50,000
Montgomery Co., Va.	G.	School	7,000
Salem, Va.	G.	Water system	11,000
Marion Co., Ill.	L. & G.	School	63,000
Stanton Co., Kan.	L. & G.	School	66,000
Columbus, Ohio	L. & G.	Concrete viaduct	64,000
Christiansburg, Va.	L. & G.	Sewage system	100,000
Boulder Co., Colo.	G.	Court house impr.	10,400
Moorhead, Minn.	G.	Steam plant add.	28,000
Lee Co., N. C.	L. & G.	Schools	60,000
Effingham Co., Ga.	L. & G.	School	21,400
George Co., Miss.	L. & G.	School	40,000
Earlsboro, Okla.	G.	Gymnasium-auditorium	3,000
State of Indiana	G.	Bridges	14,300
Alfalfa Co., Okla.	G.	Highway bridge	400
Jackson Co., Mo.	L. & G.	Waterworks impr.	29,000
Owen Co., Ind.	G.	Bridges	17,100
Rollette Co., N. D.	L. & G.	School	16,200
Lake Geneva, Wis.	L. & G.	Streets	10,000
State of Indiana	G.	Bridges	17,000
Foster Co., N. D.	L. & G.	School	75,000
Louisiana, Mo.	L. & G.	Schools	87,000
St. Louis, Mo.	G.	Streets	19,700
Salina, Kan.	G.	Streets	12,000
Emporia, Kan.	G.	Water system ext.	8,300
Hartford City, Ind.	L. & G.	Sewage plant	117,000
Columbus, Ohio	G.	Dormitory	48,000
Nashville, Tenn.	L. & G.	Incinerators	100,000
Albany, Ga.	G.	Abattoir add.	3,000
Union County, N. C.	L. & G.	Schools	105,000
Suffolk County, N. Y.	G.	Road	53,000
Montpelier, Ohio	G.	Power	12,600
St. Paul, Minn.	G.	Street repairs	155,000
Waterloo, Ala.	L. & G.	Electric system	8,100
West Chester, Pa.	G.	Waterworks impr.	49,000
Cambridge Springs, Pa.	L. & G.	Sewage	25,000
Coffeyville, Kan.	G.	Waterworks impr.	11,700
Binghamton, N. Y.	G.	Bridges	53,900
Warrenton, Va.	G.	School	20,500
Nashville, Tenn.	L. & G.	Repairs to police station	50,000
Cherokee, Okla.	L. & G.	School	32,000
Walbridge, Ohio	L. & G.	Waterworks impr.	13,000
Mott, N. D.	G.	Storage shed	1,600
Morganfield, Ky.	L. & G.	Waterworks impr.	23,000
Tiverton, R. I.	L. & G.	School	135,000
State of Connecticut	G.	Bridge and highways	79,100
Logan, Ohio	L. & G.	Waterworks impr.	94,000
Harrisonburg, Va.	G.	County jail	10,600
Littleton, W. Va.	L. & G.	Waterworks system	36,000
State of Connecticut	G.	Bridge and road	96,300
Lawrence, Kan.	G.	Museum repairs	10,500
Winfield, Kan.	G.	Ward bldg.	31,000
Wichita, Kan.	G.	Art museum	23,900
Stillwater, Okla.	G.	College impr.	19,500
Burkeville, Va.	L. & G.	Waterworks system	47,000
Wyomissing Hills, Pa.	L. & G.	Street impr.	12,000
Mower Co., Minn.	G.	Roads	33,500
Lunenburg Co., Va.	G.	Schools	16,300
Muscatine Co., Iowa	G.	Bridge	24,000
Suffolk, Va.	G.	School	4,100
Peoria Co., Ill.	G.	Sewage plant add.	25,000
Quinton, Va.	G.	Schools	16,900
Warwick Co., Va.	G.	School	8,900
Orange, Va.	G.	School add.	6,200
Clarksville, Va.	G.	School	13,000
Odebolt, Iowa	G.	Waterworks impr.	3,700
Emporia, Kan.	G.	Addition to pool	350
Decatur, Ill.	G.	Streets	36,000
McDonough Co., Ill.	G.	School	27,000
Monsanto, Ill.	G.	Roads	2,200
Bluffton, Ind.	G.	Streets	5,600
Bristol, R. I.	G.	School annex	53,000
Coshocton Co., Ohio	G.	Highway bridges	13,400
Rockingham Co., Va.	G.	School addition and heating system	34,000
Chickasha, Okla.	L. & G.	School	140,000
		Total	$4,814,837

project of the Public Works Emergency Housing Corporation.

In Duluth, Minn., in a test case relative to the CWA demolition plan, the court ruled that it was unlawful to repair buildings when they required more than 50 per cent replacement and that the demolition of such structures should be required by the building inspector.

In New York, architectural draftsmen, paid with CWA funds and working under the direction of Arthur C. Holden of the land utilization committee of the New York Building Congress, are developing what has been locally called a rehousing method of slum clearance. The plan, instead of starting with the problem of erecting new buildings, aims first at the demolition of unprofitable buildings, thereby decreasing the percentage of land coverage in depreciated areas. The blocks being studied are in the congested lower East Side of Manhattan and in Harlem.

The surveys involve not only land coverage, relative size of buildings, vacant land, etc., but also the physical and mechanical condition of the buildings, and the income and outgo, including operating expenses and mortgage interest, in relation to number of vacancies, rentals per room, etc. From these surveys charts and plans are prepared from which the reorganization plans are suggested. A system of group control and management is being worked out in order to prevent individual depreciated properties from dragging adjoining properties down in value.

In the CWA instructions from Washington it was stated that the structures to be demolished should be on public property, on private property where the owner has agreed in writing to turn the buildings over, or on condemned property whether condemned as unsuitable or dangerous or by mutual agreement. Buildings on property belonging to the Public Works Emergency Housing Corp. or other wholly public corporations are also eligible. It was suggested that no building over five stories be attempted except under competent engineering direction and that on all buildings full safety precautions be taken. It was also stated that all salvaged materials must be used for public or relief purposes.

Elbert A. Gibbs dies; Manager of McClintic-Marshall

Elbert A. Gibbs, vice-president in charge of operations of McClintic-Marshall Corporation, died suddenly on March 6, while visiting at Aiken, S. C.

Mr. Gibbs was born at Kasson, Minn., in 1881 and studied for a time at the University of Minnesota. In 1905 he graduated from University of California, and later became instructor in civil engineering at Cornell University. In 1907 he joined the McClintic-Marshall organization at Pittsburgh, Pa.

On August 1, 1919, he became general manager of the McClintic-Marshall Corp. in charge of all fabricating and erecting operations. He was elected vice-president and general manager of the company, with the same duties as before, in 1929.

During the war, Mr. Gibbs was with the Corps of Engineers in the A.E.F., attaining the rank of colonel. At the time of his death he was president of the American Society of Military Engineers.

A.S.T.M. Committees Hold Meetings at Washington, D. C.

WITH a registration of about 600, the spring meeting of the American Society of Testing Materials at Washington, D. C., Mar. 5 to 9, resulted in 155 committee and group sessions. The well attended meetings indicated the continuing interest in the research and standardization programs underway.

Cement and concrete

Reports of sub-committees of Committee C-1 on cement indicated that there would be an early release of the following: (1) a standard procedure in methods of chemical analysis as the result of a study of the methods used by twelve cooperating laboratories and (2) a tentative method for making plastic mortar compression tests of portland cement. There appeared to be a complete agreement of opinion on the general acceptance of the surface-area per gram unit as the measure of cement fineness, the question at present involves the standardizing of the method of testing and accuracy which can be used as the basis for definite cement specifications. The use of the 200-mesh sieve for determining fineness was referred to as out-of-date.

Suggestions advanced for specifications on a new low heat cement and a high sulphate cement were discussed and referred back for further study without committee action.

The concrete and concrete aggregate committee (C-9) will recommend that the five specifications for curing concrete which have been tentative since 1931 be advanced to standard at the annual meeting in June. These are: general curing specifications, and specifications for curing portland-cement concrete slabs with bituminous coverings, calcium chloride admixture, surface application of calcium chloride and wet coverings. Tentative specifications for the use of sodium silicate for curing concrete will also be recommended at the June meeting, subject to letter ballot approval.

Standard specifications for cement-concrete sewer pipe were recommended changed by Committee C-13 to read "non-reinforced concrete sewer pipe" with necessary revisions covering changes in details of test methods and elimination of all sizes above 24-in. which would be included in the reinforced concrete sewer pipe schedule. Strength requirements were materially increased for all sizes of pipe.

Outstanding result of the Committee C-7 meeting was the development of proposed tentative specifications for hydraulic lime. Although used extensively abroad, this material has been introduced into this country rather recently and the preparation of the specifications recognizes the increasing use of the new product.

Committee D-4 on road and paving materials took action recommending the adoption of a standard specification for calcium chloride and for the chemical test of this material. The group also announced the completion of a study of methods for testing liquid asphalt products with plans for taking final action in June.

Several important decisions were made in a series of well attended meetings of Committee A-1 on steel. Without a dissenting vote the committee recommended the adoption as standard of the existing tentative specifications covering steel for bridges (A 7 — 33 T), steel for buildings (A 9 — 33 T) and mild steel plates (A [illegible] — 33 T) with the concurrent withdrawal of the existing specifications for structural steel for bridges (A 7 — 33) and for buildings (A 9 — 33). These new specifications first published last year will provide a range of steel with 60,000- to 72,000-lb. per sq.in. tensile strength, replacing the 55,000 to 65,000 grade in use for years.

Reinforcing bar specifications were revised to further restrict the permissible weight variations. The new tentative revision provides tolerances on individual bars of 6 per cent under for bars ¾ in. in size and over; 10 per cent under for bars less than ¾ in. in diameter.

New specifications for electric-fusion-welded steel pipe for high-temperature and high-pressure service were approved by the committee for submitting to the society for approval.

A report on the quality of wrought iron has been completed by a sub-grade of Committee A-2 after months of study. The information to be presented in this valuable report will include a foreword outlining the investigation made by the committee followed by a discussion of the chemical composition and physical properties of wrought iron, its structural characteristics and microscopic examination illustrated by photomicrographs.

Federal Regulation Urged For All Transportation

(Continued from p. 364)

In regard to the first of these two subjects, the coordinator submitted two drafts of proposed legislation, each extending to highway carriers and water carriers the same degree of federal regulation as is now imposed upon the railroads.

In regard to the second, Mr. Eastman submitted to the President and Congress four bills amending existing provisions of the Interstate Commerce Act. The first of these bills, enabling the Interstate Commerce Commission to prescribe maximum as well as minimum joint rail-water rates and to establish through rail rates without regard to present restrictions of the act, Mr. Eastman urged be considered along with the motor and water-carrier bills, since, he held, all together were of the most pressing importance.

Mr. Eastman held that, from the standpoint of organization and experience, the Interstate Commerce Commission was the most natural and appropriate agency in which the new regulatory authority should be vested.

He dealt in his report with motor and water carriers, air transport and pipe lines. On the first three agencies his stand was definite, that federal regulation had been proven necessary. As to pipe lines, he made no recommendations, deferring comment pending the outcome of investigations by the Interior Department. As to air transport regulation, he held that its need "is sufficient to warrant definite recommendations in the form of a bill of proposed legislation at this time," but deferred detailed comment in recognition of the effects which measures now pending in Congress may have.

Personals

RALPH B. SLIPPY, consulting engineer and formerly city engineer, is a candidate for mayor of Waterloo, Ia.

WALTER D. BINGER, civil engineer of New York City, has been appointed a deputy commissioner of sanitation in that city in charge of engineering work.

H. C. LIGHTFOOT has been appointed engineer inspector with the Pennsylvania PWA organization and has been assigned to the new subway connection to the Delaware River bridge at Philadelphia.

ROBERT L. HAHN, formerly with the Shell Petroleum Corporation's construction forces, and more recently in the design department of the Kansas state highway commission, is now with the construction department of the latter organization.

RALPH VAIL, who supervised engineering work in construction of the Goodyear-Zeppelin dock at Akron, Ohio, has been appointed district CWA engineer for Summit County, Ohio. He succeeds Charles Costigan, recently named resident state highway engineer.

CHARLES E. BILLIN, secretary of the Engineers Club of Philadelphia since 1920, has resigned from that office. Mr. Billin was strongly instrumental in organizing the Philadelphia Engineers Club in 1877, and was appointed its first secretary and served in that capacity until 1879.

Obituary

THOMAS A. SHERIDAN, head of the contracting firm of Sheridan-Kirk Contracting Co. of Cincinnati, Ohio, died in that city on March 5. He was 78 years old.

THOMAS B. BRIGHTON, professor of metallurgy and head of the metallurgical department at the University of Utah, died on March 5 of pneumonia at the age of 47.

JOHN F. HUTCHINGS, a general contractor at Muncie, Ind., for many years and former city councilman, died during the first week in March. Mr. Hutchings was the contractor for steel construction of the French Lick Springs Hotel and erected many of the buildings at the St. Louis World's Fair.

SIDNEY SMYTH, contractor and engineer of Portland, Ore., died in that city last week at the age of 69. Mr. Smyth was born in New Hampshire but had been living in the West since 1890. He had the first contract for the locks in the Celilo canal and also put in Seattle's original water system. At the time of his death he was consulting engineer for the Los Angeles Flood Control District and the Northwest Roads Co.

GEORGE A. GREEN, marine construction engineer of Green Bay, Wis., and former city commissioner, died recently as the result of carbon monoxide poisoning in the garage at his home. Mr. Green had suffered an automobile accident about a year and a half ago from which he had never fully recovered. At the time of his death he was 55 years old. Some of the more important work done by Mr. Green's contracting company were the laying of submarine pressure mains under the Mississippi River at Burlington, Iowa, and at East St. Louis, Ill., and construction of water intakes at Hammond, Ill., and at Waukegan, Ill.

CONSTRUCTION STATISTICS OF THE WEEK

ENGINEERING CONTRACT awards drop this week to $18,993,000 from last week's high of $34,023,000. This total is below the year's average to date, below the previous 4-weeks' average, but 64 per cent above the average March week last year. Private work awarded this week, $4,332,000, is above the average week to date this year if last week's high total of $18,625,000 is excluded. Public work dropped to $14,661,000 from $15,398,000 and is below the average week to date, $18,300,000. Federal awards are slightly above last week's but state and municipal awards drop below.

This drop is principally in streets and roads awards which are down to $4,452,000 compared with $7,879,000 last week and $7,700,000, average week to date. Sewerage contracts are above last week, $1,194,000 compared with $726,000; bridges are up, $1,492,000 compared with $1,122,000; earthwork, irrigation and drainage is down, 1,407,000 compared with $7,500,000 last week. Industrial buildings are way down, to $1,647,000, from last week's record total of $12,313,000; commercial buildings are up, from $890,000 to $2,505,000, and public buildings are way up from $2,336,000 last week to $5,428,000 this week.

Large projects this week include a department store annex in Indianapolis, $1,500,000, buildings at Pilgrim State Hospital, Dept. of Mental Hygiene, New York, $2,114,000; officers' quarters at Patterson Field, Fairfield, Ohio, $1,018,000; highway letting, Montana, $1,118,000; and bridge superstructure over Fore River, Mass., $717,000.

New capital this week is dominated once more by PWA allotments, $5,627,000 to municipalities and $2,462,000 to corporations and railroads. Allotments for limited dividend housing projects aggregating $23,670,500 have been rescinded and this amount transferred to the Federal Housing Corporation. The municipal bond market continues strong but the issues sold this past week were principally refunding, welfare, relief and soldiers' bonus. Sales of only $1,007,000 in bonds for construction were recorded. Bond elections scheduled and bond offerings for construction authorization continue active.

CONTRACTS
(Thousands of Dollars)

	Weekly Average Mar. 1933	Prev. 4 Weeks	Week Mar. 15 1934
Federal Government	$2,538	$3,929	$3,720
State and municipal	4,373	12,454	10,941
Total public	$6,912	$16,383	$14,661
Total private	4,675	7,848	4,332
Weeks' total	$11,587	$24,231	$18,993

Cumulative to date:
1933.....$192,914 1934.....$258,089

NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

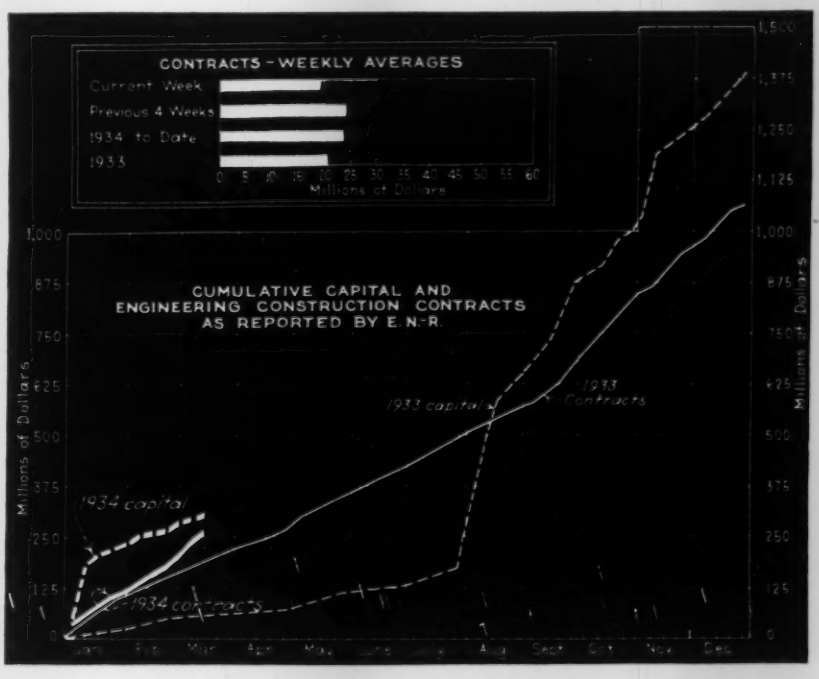
1934	Week Mar. 15	Cumulative
State and municipal	$1,007	$50,294
PWA allotments, S&M	5,627	194,446
Corporate issues	....	10,525
PWA allotments, private	2,462	44,581
Total, Non-Federal	$9,096	$299,846
PWA allotments Federal Constr.	851	48,102
Total new capital	$9,947	$347,948

Cumulative to date:
1933.....$64,300 1934.....$299,846
Rescinded or reduced allotments (from 1933 PWA totals)..... $43,323

INDEX NUMBER

E.N.-R.-Cost		E.N.-R.-Volume	
March, 1934	194.04	February, 1934	88
February, 1934	194.04	January, 1934	113
March, 1933	158.44	February, 1933	82
1933 (Average)	170.18	1933 (Average)	102
1932 (Average)	156.97	1932 (Average)	127
1931 (Average)	181.35	1931 (Average)	220

1913 (Average).....100



Current Construction Unit Prices

Six Close Bids on Cleveland Sewage Plant

SIX UNUSUALLY close bids were received from Cleveland contractors for the construction of the concrete sludge digestion tanks at the Easterly sewage treatment works at Cleveland. Bids were opened Nov. 2. Proposals were submitted by the following: Hunkin-Conkey Const. Co., $1,091,119; Highway Const. Co., $1,097,223; National Engineering & Contracting Co., $1,138,780; Dawson & Finan, $1,160,004; Lundoff-Bicknell Co., $1,193,010; Jos. Witerbottom Co., $1,323,794 all based on stone or gravel aggregate for concrete.

Work will be under the direction of Thos. M. Carey, director of public utilities. Robert Hoffman is consulting engineer; George B. Gascoigne is consulting sanitary engineer. Following are unit prices on principle items only, as submitted by the three lowest bidders. (A) Hunkin-Conkey Const. Co.; (B) Highway Const. Co.; (C) National Engineering & Contracting Co. Contract has been awarded to Hunkin-Conkey.

UNIT PRICES ON PRINCIPAL ITEMS—SLUDGE DIGESTION TANKS EASTERLY SEWAGE TREATMENT WORKS, CLEVELAND

Item	Quan.	Unit Prices A	B	C
Excavation, c.y.	46,000	$1.10	$1.55	$2.10
Wood piling, lin. ft	205,000	.36	.60	.48
Concrete (stone or gravel aggregate) c.y.	18,100	14.13	13.46	14.44
Concrete (slag aggregate) c.y.	18,100	13.58	13.00	14.00
Steel reinforcement, tons	2,420	60.38	62.00	64.00
Expansion joints, lin. ft	1,850	1.68	2.00	1.67
B. & S. C. I. pipe, tons	240	80.00	71.00	76.00
Flanged C. I. pipe, tons	5	106.00	112.00	96.00
B. & S. C. I. fittings, tons	6	138.50	137.00	140.00
Flanged C. I. fittings, tons	72	183.30	149.00	175.00
Special C. I. fittings, tons	37	150.30	152.00	150.00
Chrome-nickel iron pipe, 2 in., l.f.	2,800	.84	.66	.55
Chrome-nickel iron pipe, 3 in., l.f.	2,800	1.32	1.11	.90
Gate or check valves, 8 in., each	97	65.50	59.20	61.00
Gate or check valves, 10 in., each	44	134.30	118.00	121.00
Vault lights, sq. ft	9,200	2.45	2.30	2.30
Buildings, lump sum	.....	31,100	37,000	32,000
Sludge digestion equipment	.....	267,343	230,000	260,000
Sludge heating equipment	.....	95,385	83,000	86,000
Electric wiring, lump sum	.....	22,914	21,500	20,000

Test Borings, Ft. Peck Project, Montana

BIDS WERE OPENED Oct. 31 at the Kansas City District Engineer Office for test borings and test pit work on the Ft. Peck project, Missouri River, Montana. Seven bids were submitted, ranging from $37,664 to $72,281. Following are complete unit prices of the three low bidders. (A) Sam J. Mathews, Tulsa, Okla.; (B) Sprague & Henwood, Inc., Scranton, Pa.; (C) The Mott Core Drilling Co., Inc., Huntington, W. Va. Capt. Theo. Wymann, Jr, is district engineer in charge for the War Department.

TEST BORINGS AND TEST PITS, FT. PECK PROJECT

Item	Quan.	Unit Prices A	B	C
Overburden drilling, 52 holes, l.f.	2,560	$2.50	$3.00	$2.60
Core drilling, 0 to 50 ft., 3 in., 128 holes, l.f.	3,210	2.50	3.00	3.10
Core drilling, 50 to 200 ft., 3 in., 44 holes, l.f.	4,115	2.50	3.00	3.35
Core drilling, over 200 ft., 3 in., 10 holes, l.f.	265	2.50	3.00	4.00
Casing left in holes, l.f.	2,935	.40	.25	.65
Open test pits, 6 pits, l.f.	420	12.50	16.00	13.00
Ouwatering test pits, 4 pits, l.f.	50	20.00	10.00	8.00
Total bid		$37,664	$44,669	$44,835

Three New Hampshire Highway Projects

BIDS OPENED by the New Hampshire highway department on Nov. 3 for three road projects are typical of the prices obtaining on highway work in that state last fall under the PWA regulations. Unit prices submitted by low bidders (contract) on each of the three projects are given in the following table. Projects and bidders are as follows: (A) NRM project in Rochester, S. E. Rand Transportation Co., Portsmouth; (B) NRH project in Milford, Central Const. Co., Lawrence, Mass.; (C) NRH project in Littleton, Central Const. Co., Lawrence, Mass.

PRINCIPAL ITEMS ON THREE HIGHWAY PROJECTS, NEW HAMPSHIRE

Item	Low Bids on Projects A	B	C
Earth excavation, cu.yd.	$0.48	$0.40	$0.55
Fine grading, sq.yd.	.03	.01	.03
Gravel base course, cu.yd.	.50	.45	.90
Stone base course, ton	1.90	2.40	...
Bit. macadam surface course, ton	1.90	...	...
Apply bit. material for bit. mac., gal.	.10	...	...
Furnish and apply ref. tar, gal.	.14	...	...
Reinf. conc. pipe, 12 in., l.f.	1.00	1.00	1.10
Reinf. conc. pipe, 15 in., l.f.	1.50	...	1.45
Reinf. conc. pipe, 24 in., l.f.	3.00	...	...
Reinf. conc. pipe, 18 in., l.f.	...	...	1.90
Vit. clay pipe, 6 in., l.f.	.60	...	.60
Vit. clay pipe, 12 in. l.f.	...	.80	...
S. C. Premix bit. macadam, ton	...	6.10	...
Reinf. Conc. pavement, sq.yd.	...	...	2.25

Lock 16, Mississippi River, Typical of Several

WITH ALLOTMENTS from the Public Works fund, army engineers are rushing the upper Mississippi River navigation control program into the construction stage. A total of 26 lock and dam developments is planned for the river between St. Louis and St. Paul. Of these, two have been completed, and a third is well along. Thirteen additional locks will be placed under contract this season. Typical of these is Lock 16, near Muscatine, Ia., contract for which was let early in November. Three bids were received, ranging from $1,196,979 (contract) to $1,844,896. Work consists of a main lock 110x600 ft. and an auxiliary lock 110x380 ft., with guide and guard walls of a maximum length of 1,940 ft. Major R. A. Wheeler, district engineer at Rock Island, will be in charge.

Following are complete unit prices submitted by the three bidders. (A) Central Engineering Co., Davenport, Ia., contract; (B) Maxon Const. Co., Inc., Dayton, Ohio; (C) Northern States Contracting Co., St. Paul.

COMPLETE UNIT PRICES—LOCK 16, MISSISSIPPI RIVER

Item	Quan.	Unit Prices A	B	C
1. Cofferdam, lin. ft	937	$122.50	$100.00	$221.00
2. Excavation, common, c.y.	66,000	.32	.65	1.50
2. Fill and backfill, c.y.	38,660	.37	.30	.25
4. Timber piling, lin. ft.	199,580	.50	.50	.60
5. Steel sheet piling, sq. ft.	92,753	1.12	1.30	1.50
6. Timber cribbing, f.b.m.	402,800	50.00	100.00	80.00
7. Timber, oak, f.b.m.	24,130	100.00	150.00	220.00
8. Stone, derrick, c.y.	1,718	3.45	4.00	6.00
9. Stone, riprap, c.y.	6,502	3.00	4.00	4.00
10. Concrete, class A., c.y.	395	15.00	20.00	25.00
11. Concrete, class B, c.y.	65,976	8.07	10.00	13.85
12. Structural steel, lb.	1,706,370	.062	.054	.06
13. Reinf. steel, lb.	640,975	.035	.04	.05
14. Steel, corrosion-resisting, lb.	13,078	.44	.40	.41
15. Steel, struct. nickel, lb.	10,550	.22	.19	.20
16. Steel, nickel-chrome, lb.	456	.62	.40	.65
17. Forgings, steel, lb.	246,673	.045	.09	.09
18. Steel castings, lb.	100,291	.10	.102	.10
19. Alloy steel castings, lb.	5,220	.15	.158	.15
20. Iron castings, lb.	30,788	.08	.069	.07
21. Bronze, lb.	4,050	.45	.42	.44
22. Copper, lb.	215	.50	.42	.44
23. Nickel-copper alloy, lb.	1,028	.40	.66	.32
24. Misc. non-ferrous metals, lb.	2,802	.20	.25	.17
25. Brass pipe, ¼ in., lin. ft	327	1.00	1.00	1.05
26. Blk. steel pipe, 6 in. l.f.	1,650	1.58	2.00	1.75
27. W. I. pipe, 2 in., lin. ft.	517	.60	1.00	1.63
28. C. I. pipe, 4 in., lin. ft.	14	2.00	1.00	2.50
29. Vit. pipe, 10 in., lin. ft.	654	1.40	1.00	.80
30. Vit. pipe, 12 in., lin. ft.	516	1.90	1.00	1.05
31. Fiber conduit, 2 in., l.f.	5,700	.20	.20	.30
32. Fiber conduit, 3 in., l.f.	50,700	.23	.25	.33
33. Fiber conduit, 4 in., l.f.	135	.35	.40	.43
34. Metal conduit, 1 in., l.f.	114	.33	.30	.21
35. Metal conduit, 1¼ in., l.f.	94	.60	.50	.35
36. Metal conduit, 2 in., l.f.	570	.80	.50	.53
37. Metal conduit, 3 in., l.f.	4	2.00	1.00	1.60
38. Steel pipe handrail, l.f.	1,661	1.65	1.95	3.40
39. Rubber seals, lb.	4,810	.85	.65	.80
40. Tile gages, l.f.	138	5.20	3.00	6.80
41. Street washers, 2 in., each	4	25.00	50.00	53.00
42. Gate valves, 4 in., each	2	20.00	40.00	26.00
43. Lock gate oper. mach., each	4	5,240	4,500	5,000
44. Tainter valve oper. mach., each	4	3,605	3,200	4,000
45. Central control station, compl.	1	16,130	20,000	22,000
Total bids: A—$1,196,979; B—$1,354,184; C—$1,844,896.				